WATER POWER

A technical journal devoted to the study of all aspects of Hydro-electric developments

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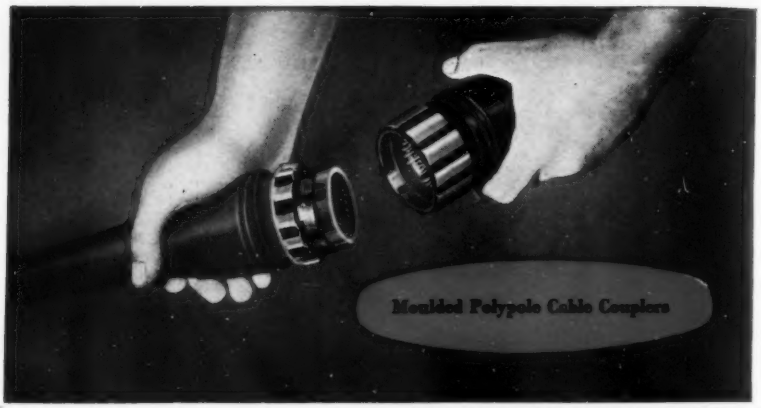

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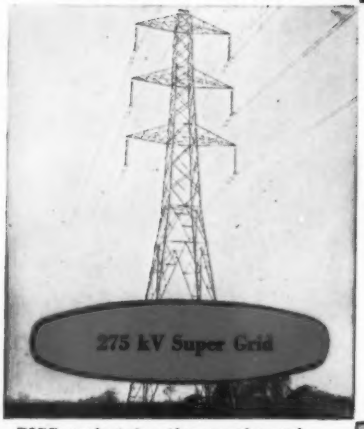

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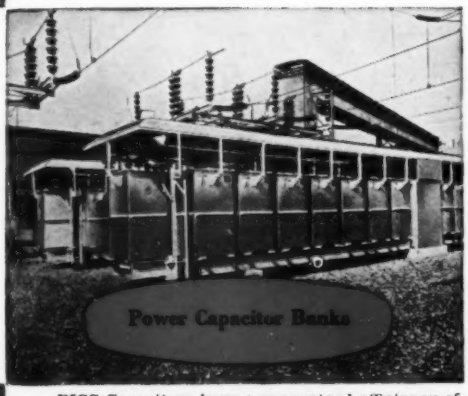

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WATER POWER

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The Geneva Conference

THE first International Conference on the Peaceful Uses of Atomic Energy, held in August in Geneva, proved to be not only a focal point for the bringing together of an enormous diversity of information about atomic power problems, but also provided an opportunity for assessing world power requirements. The Papers surveying the estimated world needs in the years 1975 and 2000 showed that there is still a wide variation between such estimates; and they also showed that experts still disagree on the amount of potential water power remaining to be harnessed.

The Conference, in addition to discussing atomic energy, provided an occasion for a survey of alternative power resources of all kinds. The utilisation of solar energy has been much in the news in the current year, particularly in view of the Paper presented by Sir Harry Railing at the British Electrical Power Convention in July, in which he suggested that since the world population may well increase from 2·33 billion in 1950 to 2·9 billion in 1975, and to 6 billion in the year 2050, we should rapidly reach the stage where dependence on "earthly survival will rely wholly on solar energy, and it will be necessary that about 4 per cent. of the solar energy falling on land should be converted for man's use at an efficiency of about 10 per cent."

The papers at Geneva clearly showed that it is only in a few countries, where the whims of nature have effected such a random distribution of natural resources that industrial demand has outstripped the rate of coal extraction or the possibilities of water-power development, that atomic energy must of necessity play a vital part in the next two or three decades. Other countries have no need, for many years to come, to embark on the large expenditures necessary to institute a nuclear power programme.

At the present time only three countries are capable of providing nuclear fuels in any quantities—the United States of America, Great Britain, and the U.S.S.R. Any other nation which wished to build any but the simplest reactor, of modest size, would have to become, for many years, entirely dependent on one of these three countries for their fuel-element processing work.

Nevertheless, it was clearly obvious that everyone at the Conference, which had attracted representatives from practically every country in the world, felt that atomic power would be an important source of energy in the next century, whatever may be the outcome of experiments in harnessing radiations from the sun.

However, as we have said before in these columns, whatever may happen in regard to the development of new sources of electrical power and the virtual abolition of the employment of coal and oil in conventional power stations, no authoritative spokesman from any country, or connected with any major organisation, has ever predicted the state of affairs where new water power will no longer be harnessed, or where existing water-power stations will no longer be required to operate. In all the discussions about the future of energy both in regard to the harnessing of new sources or the utilisation of the present methods, water power is the one constant immutable factor, and nothing emerged from the Geneva Conference to suggest otherwise.

There is, however, another factor to consider. Increased power availability is intended to bring an improved standard of living, and when this is sought after and achieved it invariably brings in its train two other needs in regard to the control and utilisation of water resources. Higher living standards imply more food consumption per head of the population, and more food can only be produced, in the vast majority of cases, by better irrigation. Secondly, the areas bordering on many of the great rivers of the world have for centuries formed the chosen sites for agglomerations of towns and villages, whose inhabitants need the river waters for cultivating their land, for domestic purposes and for navigation; and in a large number of cases these areas are subject to periodical inundation by flooding. Therefore, when control of water for irrigation, flood prevention and

improvement to navigation are under consideration, it is obvious that the opportunities for power generation created by the erection of dams and barrages will always be seized. In consequence, the atomic power station, by raising the general standard of power availability, and consequently creating a world-wide demand for the higher standards of living attendant on cheap and abundant power will inevitably lead to water-power developments initiated, perhaps, from different motives than those which have hitherto prevailed.

Whakamaru, New Zealand

THE Whakamaru power development, some account of which is given elsewhere in this issue, is of special importance to the economy of the North Island of New Zealand. In the South Island the potential for hydro power is abundant but in the North Island, which contains two-thirds of the country's population, the water-power resources are relatively meagre. There are few rivers of importance, the only sizable one being the Waikato, on which the Whakamaru station is situated. Fortunately this river has considerable possibilities; Arapuni, the first station to come into production on the Waikato, was started in 1925, though the last of its eight generators, which raised its capacity to 157,800 kW, was not installed until 1946. Karapiro (90,000 kW) was started in 1940, and completed in 1948, and by October 1943 a comprehensive scheme was adopted to use virtually the whole of the fall from Lake Taupo, its source, to Cambridge, about 90 miles downstream. In that distance the fall is over 1,000 ft., the total fall to the sea being about 1,200 ft.

The next station to be erected was Maraetai, biggest of all (180,000 kW), started early in 1946 and completed in May 1954. Next in succession is Whakamaru, with its construction well under way.

Ultimately it is planned to have ten power stations operating on this river. These will have a total capacity of about 800,000 kW so that the annual output from this one stream will represent the equivalent in power of more than 3,000,000 tons of coal—more than the total coal production in any one year.

British Columbia Contract

THE contract for the Daisy Lake dam on the Cheakamus River in British Columbia has been awarded to Emil Anderson Construction Company, the contract price, one of four submitted, being $2,438,355·50. The dam will be a 500,000 cu. yard earth and rockfill construction, 2,000 ft. long and 90 ft. high, and will require a 3,000 ft. diversion tunnel and a tunnel intake.

This dam will be a major component in the 190,000 h.p. $25 million Cheakamus project, which was commenced in 1954 and is scheduled for completion in 1957. It is situated 23 miles north of Squamish and will convert Daisy Lake and the surrounding swamp land into a 1,050 acre storage basin.

It is the second construction contract to be let on this scheme, the first having been awarded to Mannix Construction Company to drive a six-mile tunnel to connect the Cheakamus with the Squamish River. The contract for the construction of a power house at the Squamish end of the tunnel will be let before next spring. The plant will consist of two 95,000 h.p. turbines—the largest yet installed on the B.C. Electric's system. A 14-mile 230 kV transmission line will convey the power to the Bridge River circuit at Cheekeye for delivery to Vancouver. Full automatic control of the station will be exercised from the load control centre at Vancouver by carrier impulse.

A description of B.C. Electric's Bridge River plant, with particular reference to the 62,000 h.p. six-nozzle vertical turbines, was published in our September, 1953, issue, page 343.

New Powell River Line

PLANS for a 4½ million dollar power line between Squamish, 20 miles north of Vancouver, and Powell River, across 100 miles of mountainous terrain, have been announced by British Columbia Electric. The 132 kV line, designed to ease the hydro shortage of the thriving papermaking township, will also supply domestic users who, because of power limitations, have been unable to operate electric ranges and other equipment. The line will also furnish additional power to the Powell River Company's mill and should be completed by the end of 1956.

Six-year Plan for Portugal

AN extra £3 million is being set aside for the hydro-electric power station at Picote (River Douro) under the revised six-year plan estimates, and over £3 million to meet the increased cost of the stations at Canicada and Paradela. Transmission lines will absorb a further £2½ million, and local distribution systems an extra £2 million. Considerable sums will also be spent on auxiliary thermal stations, which will be designed to consume the lowest grades of Portuguese coal.

Litani River Project

TO meet the power shortage in Lebanon and to increase the acreage under cultivation the World Bank has arranged credit facilities to the amount of $27 million, the major works envisaged being a hydro-electric plant on the Litani river. There has been an acute shortage of power in Lebanon, especially in Beirut, since 1945, and the new generating plant will double the present generating capacity. In addition, the storage impounded by the dam will enable some 8,500 acres to be irrigated along the Mediterranean coastal region and so enable increased food supplies to be grown both for home consumption and export. The scheme is being entrusted to the Litani River Authority, which was formed under the aegis of the Lebanon Government last year.

Swiss Hydro Progress

SWITZERLAND recorded in June a substantial advance in hydro-electric generating capacity. On the 21st, the Les Clées II plant, designed for a mean yearly production of 94 million kWh, was put into commission, and nine days later, the 32 MW Ernen power house (108 million kWh in summer, 57 million kWh in winter) was officially opened. At about the same time, two 28 MW generating groups were put into service at Cavergno, increasing to 500,000-600,000

kWh the aggregate daily production of the Maggia installations, which, for the time being, are still operating as run-of-river plants. In addition, work began on the 30 MW Pallazuit power house, belonging to the Société des Forces Motrices du Grand-St-Bernard; this plant will, upon completion, produce a yearly average of 82 million kWh, of which 31 million kWh in winter.

Mourne Scheme

THE Northern Ireland Ministry of Commerce has introduced a hydro-electric scheme which will cost £3,750,000 to develop the river Mourne and its tributaries for the generation of electricity. Construction of a dam with an 18 MW hydro-electric station near Mulvin, and of a dam with a 9½ MW hydro-electric station near Sion Mills, is envisaged. The scheme also provides for the electrical interconnection of the stations and for their connection with existing power sources. The combined stations should be able to produce about 78 million kWh annually, which is more than 9 per cent. of the quantity sold in Ireland in 1954. The annual cost of producing these extra units is estimated to be some £100,000 less than their cost of production in a modern coal-fired station with fuel at its present price. One object of the scheme is to extend the main high-voltage transmission system from its present terminus at Omagh to the new hydro stations. At the moment the principal generating stations are on the East Coast.

Noxon Rapids Turbines Ordered

A CONTRACT for four 137,500 h.p., 152 ft. head hydraulic turbines has been awarded Allis-Chalmers Manufacturing Company by the Washington Water Power Company through its agent, Ebasco Services, Inc., for its Noxon Rapids hydro-electric development on Clark Fork in north-west Montana. The turbines will be of the vertical Francis type and will have welded plate-steel scroll cases. Twin cabinet-type oil-pressure governors will be provided. In physical size, it is stated that these runners will rank with the largest in the world.

The first turbine is scheduled to be shipped during the latter part of 1958, with the others following at about three-month intervals. The runners will be designed in several sections to facilitate transport.

Rotorua Scheme, New Zealand

DRILLING is at present being carried out along possible routes for tunnels between lakes in the Rotorua district. This represents a first step in an investigation as to the practicability of coupling the major lakes as storage capacity for a proposed hydro-electric scheme on the Kaituna River. The drilling is purely investigational and so far the results have been very favourable. Completion of these tests might take a year, by which time it is expected that all possible routes will have been surveyed. Should inter-lake tunnelling then be considered economic, practical operations would be started. Simultaneously, investigations are proceeding along the river itself, where it is proposed to establish two, and possibly three, power stations. In 15 months the line of a 4¾-mile diversion tunnel from Otaramarae Bay on Lake Rotoiti has been tested and more than a dozen bores have been sunk in the bay itself and down the east bank of the river. The Kaituna could not be expected to produce power before 1962, but it could, it is thought, relieve the burden on the Waikato Grid to the extent of 90,000 kW.

Colombia Plan

THE sum of 500,000 pesos for the financing of preliminary survey work has been allotted by the Colombia Government as a first step towards industrialisation of the Cauca Valley, West Colombia. The construction of reservoirs and dams for flood control and the production of hydro-electric power are envisaged by the plan. It is possible that a representative of the International Bank for Reconstruction and Development will take part in this survey.

Aerial Survey of British Guiana

AS part of the Government's policy to aid Colonial development, the Directorate of Colonial Surveys is undertaking a further aerial survey of British Guiana, the contract for which has been awarded to Hunting Aerosurveys Limited. The 54,000 square miles of British Guiana yet to be surveyed include some of the wildest and most unexplored country in the world. Dense and virtually uninhabited forests cover over 80 per cent. of the country and most areas are almost inaccessible except by air, as frequent rapids greatly limit the use of rivers.

The aerial photography will be used in the first place to provide maps which are the essential forerunners of any industrial or economic programme. One project which has often been considered is the development of the spectacular Kaieteur falls, with its sheer drop of 750 ft., as a source of hydro-electric power. Another is the mineral potential of the Pakaraima Mountains where considerable mineral possibilities are known to exist.

Thomas Lawrie, C.B.E., F.R.S.E.

IT is with great regret that we record the death of Mr. Thomas Lawrie, general manager of the North of Scotland Hydro-Electric Board, at the relatively early age of 54. After graduating at King's College, Cambridge, in 1923, he spent some years with The English Electric Co. Ltd., at Stafford, and subsequently went to the United States. His experience in the hydro-electric field was, however, gained in this country as secretary to the Power and Traction Finance Corporation which promoted electrification schemes in various parts of the world. He took an active part in promoting the Galloway scheme and at a later date joined the North of Scotland Hydro-Electric Board as secretary.

In a tribute to his qualities Mr. Thomas Johnson, chairman of the Board, referred touchingly to Mr. Lawrie's capacity for hard work, for his appreciation of the finer details of administration, for his courtesy and urbanity, for his skill in negotiations and for the manner in which he was able to secure the wholehearted co-operation of his staff.

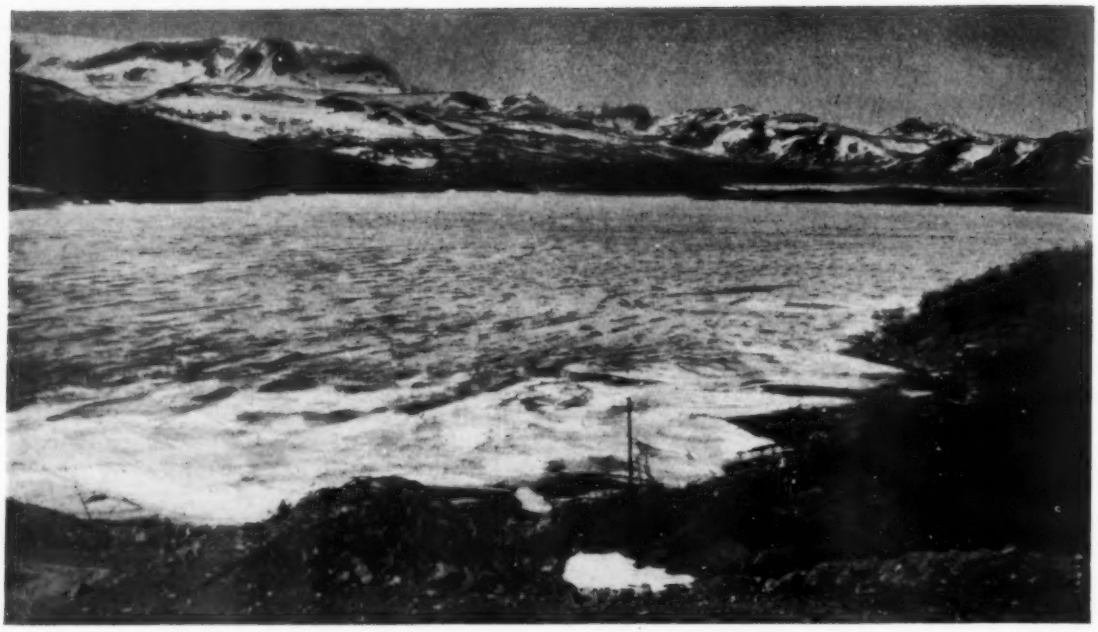

Fig. 1. Strandevann, the largest storage on the Urunda, before impounding

The Hol Developments

An important access to Oslo's power supply is being derived from developments on the Hallingdal river and its tributaries which will ultimately have a capacity of about 800 MW. The first of these developments is partly in commission and contains the second highest-head Francis turbines in operation in Norway. A 220 kV line conveys the power 187 km. to Oslo

PART ONE

RUUD power station, which is the first of a series of developments on the Hol-Hallingdal river in the Norwegian county of Buskerud, is the second hydro-electric station owned entirely by the city of Oslo. The first municipally-owned plant—the Hammer station—was built as long ago as 1900 on the Akers river, but was reconstructed in 1927, when six small machines were replaced by a 6,000 h.p. horizontal Francis turbine coupled to a 5,000 kVA generator.

In 1924 a station was brought into operation at Solbergfoss on the river Glomma, this station being owned jointly by the Norwegian Government and by Oslo Lysverker (Oslo's Electricity Department) in the proportions of one-third and two-thirds respectively. An agreement has also been reached with A/S Glomma Traesliberi to purchase a permanent block of power from their Kykkelsrud station, and to provide reserve and peak power Oslo Lysverker has built a 25 MW steam station.

Thus at the end of 1948—immediately before the Ruud station commenced operation—Oslo had at its absolute disposal the following hydro-electric capacity: Hammer 4 MW, Kykkelsrud 17 MW and Solbergfoss 45 MW, making a total of 66 MW. Adding the capacity of the steam station this represented a total of 91 MW under the control of the municipality.

By contrast, in 1948/49 Oslo's maximum demand was 235 MW and the annual consumption 1,520 million kWh, and the balance was made up principally by purchasing power from Government-owned plants and partly through the Joint Electricity Supply Organisation for East Norway (Samkjøringen).

The Hol Catchment

To meet this situation Oslo Lysverker had already acquired extensive water rights in the Hol catchment in Upper Hallingdal and in the adjacent catchment in Western Norway containing the headwaters of the

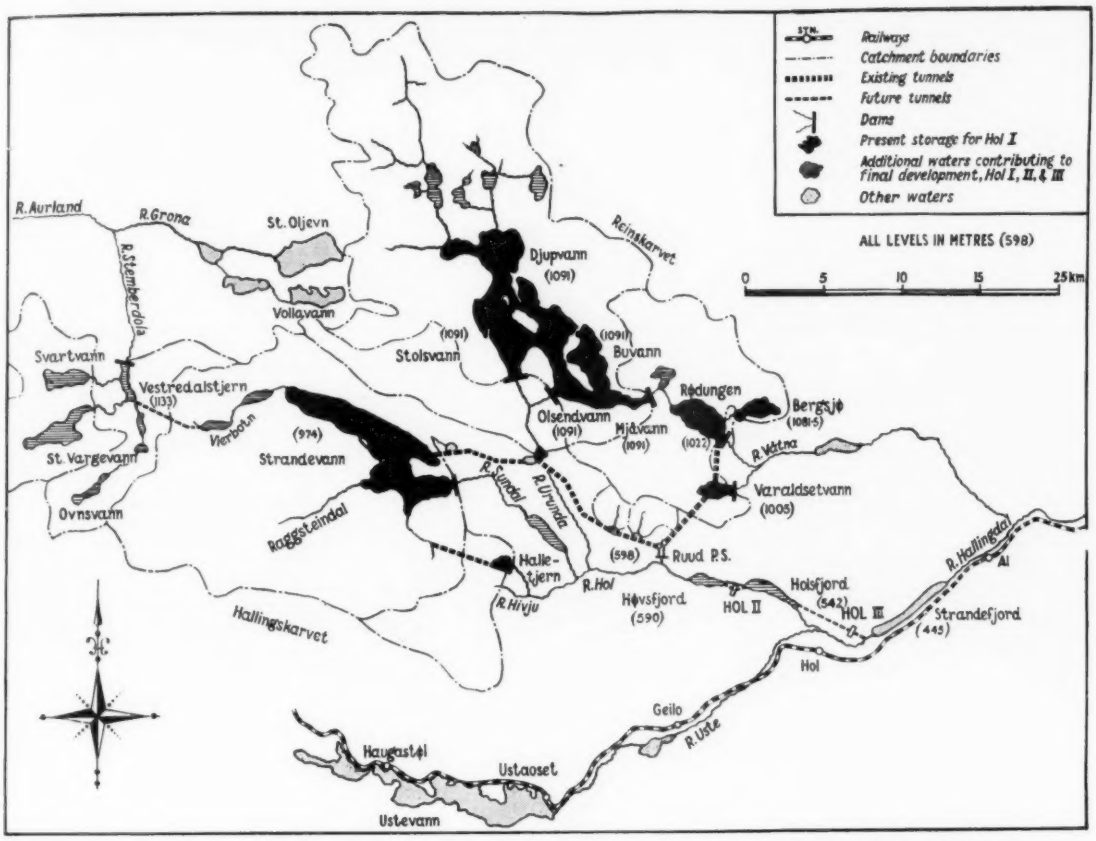

Fig 2. Map of the Hol catchment, showing the locations of Hol I, II and III developments

Aurland river. This area is roughly 190 km. north-west of Oslo and is particularly attractive for hydro-electric development both for its high power potential and for the extensive high-level storage it affords. Storage is a particularly important factor because the flow in the river system is greatest in spring and early summer, when the mountain snows are melting, whereas the peak power demand is during the winter. In the Hol development an ingenious use of the catchment area has enabled nearly 100 per cent. regulation to be obtained.

The river system can be studied with the aid of the map of the catchment area given in Fig. 2. The drainage area is bounded on the south by the Hallingskarvet range, which rises to 1,933 m., and on the north-east by the Reinskarvet range. To the west are the headwaters of the Aurland river, which flows west, and to the east are those of the Votna, which flows east and joins the Hallingdal below the village of Ål. The Hol drainage has an area of 595 sq. km. and a mean annual runoff of 1,150 mm., the corresponding runoff in the Aurland basin to the west of the watershed being 1,700 mm. The Votna drains an area of about 130 sq. km.

To the south of Hallingskarvet is the basin of the Uste, which derives from Ustevann and flows north-east to meet the Hol below Holsfjord.

The Hol river runs eastwards and is fed by three main tributaries—the Hivju, the Sundal and the Urunda. Most of the flow is derived from the Sundal and the

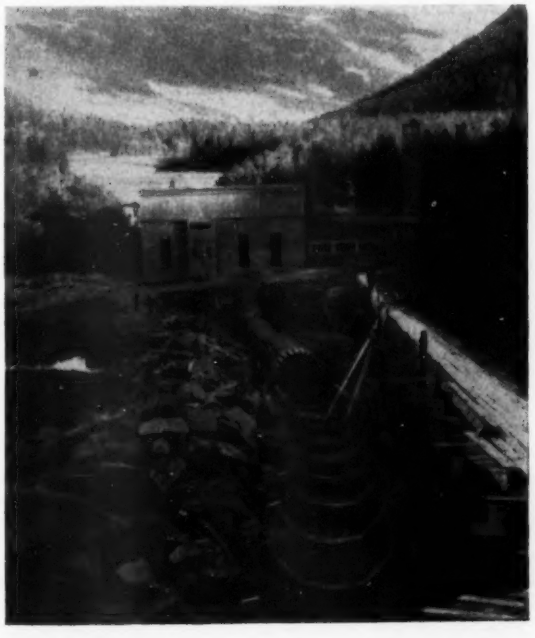

Fig. 3. The temporary station at Djupedal under construction

Urunda, as the Hivju has its source in a minor lake, the Halletjern at an elevation of 1,082 m. The Sundal river, known in its upper reaches as the Storå, has its origin in the largest lake in the vicinity, Strandevann, which lies at 950 m. and drains an area of 263 sq. km. From this lake it descends to Sundalsfjord, lying at 731·5 m., and thence it flows into the Hol from a north-westerly direction. The Urunda lies to the east of the Sundal and roughly parallel to it, and is fed by a network of lakes of which Djupvann (1,084·5 m.), Stolsvann (1,079 m.) and Olsennvann (1,084 m.) are the most important.

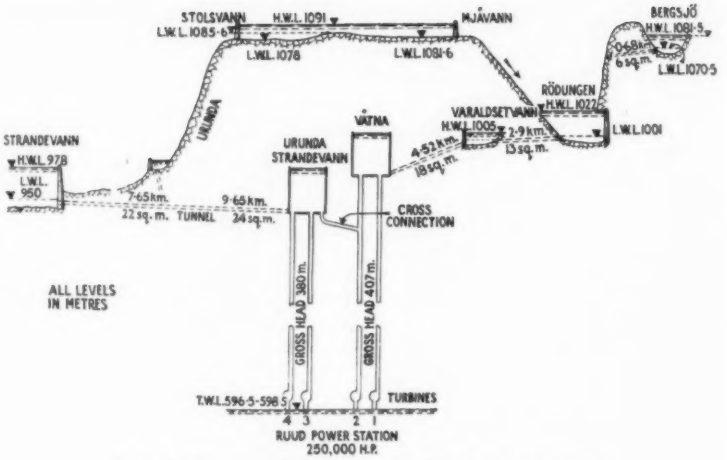

Fig. 4. Storage diagram of the Hol I development

The Hol, so-called, commences at the junction between the Sundal and the Urunda, and flows into Hövsfjord (590 m.), thence over the Djupedal falls to Holsfjord (542 m.), then over Hagafoss falls to the junction with the Uste, and finally (470 m.) under the name of the Hallingdal river to Strandefjord (445 m.). From the outlet from Strandefjord the river flows north-east over the Torpo and the Gol rapids to the village of Gol (beyond the map, Fig. 2) where it is joined by the Hemsila, turns south-east, and becomes the Snarum and finally the Drammen, flowing through the town of that name to discharge into Drammens-

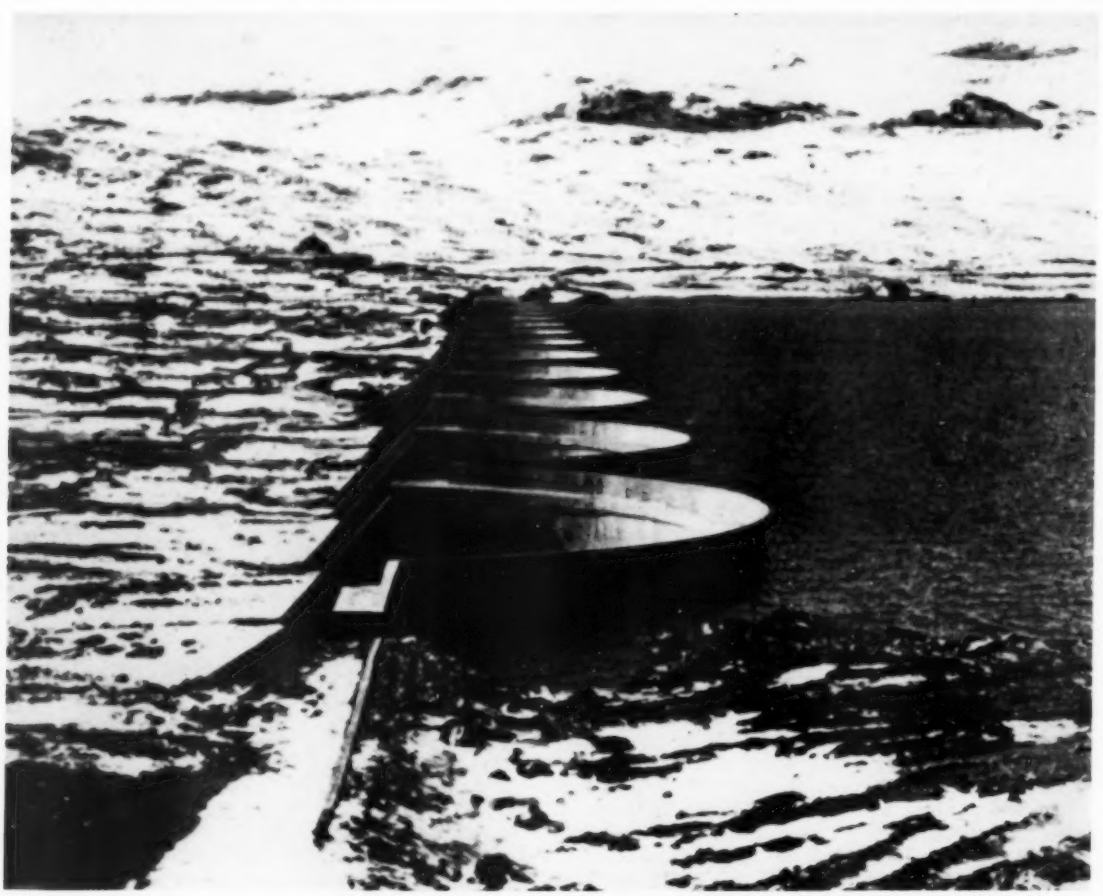

Fig. 5. Stolsvann dam, the largest controlling the high-level storage

Fig. 6. Olsennvann dam, in the high-level storage system

fjord, which is a western offshoot of Oslofjord.

In the Hol-Hallingdal valley itself four developments are projected, three of which are under construction by Oslo Lysverker and form the main subject matter for these articles. Ruud power station, to which we have already referred, constitutes the development known as Hol I and will utilise the headwaters of the Hol and the Votna together with a transfer of headwaters of the Aurland from a drainage area of 200 sq. km. Hol I will have an ultimate capacity of 190 MW and has been operating at half capacity since December, 1949. Construction on the remainder of the development is well advanced.

A temporary station was built at the Djupedal falls as long ago as 1931 and was enlarged to a final capacity of nearly 2·4 MW in 1942 and used for construction power. Hol II, now under construction, will constitute the permanent development at Djupedal and will have a capacity of 27 MW. Hol III, also under construction, will utilise the head between Holsfjord and Strandefjord and will have a capacity of 60 MW. These three stations will have average annual outputs, based on the 30 years' records from 1918-1948, as follows:

Hol I	670	million kWh
Hol II	100	,, ,,
Hol III	210	,, ,,
Total	980	,, ,,

The production given above for Hol I relates to the scheme as at present under construction. When waters from the Aurland and the Hivju have been brought in, as described later, the annual production of Hol I alone is expected to be nearly 900 million kWh.

A further development, Ål-Nes, is also envisaged to utilise the fall between Strandefjord and Nes, which will provide a head of 293 m. Ål-Nes will have the benefit of the flow from the Uste and will have a capacity of about 230 MW.

In addition to these schemes in the Hol valley proper, two stations are planned on the Uste to utilise a head of 550 m. and totalling 136 MW, and three in the Hemsila basin totalling 145 MW. Thus the fully developed capacity of this river system upstream of the confluence of the Hemsila and the Hallingdal is as follows:

Hol I	190	MW
Hol II	27	,,
Hol III	60	,,
Ål-Nes	230	,,
Uste I & II	136	,,
Gjuva Hemsila I Hemsila II	145	,,
Total	788	,,

In addition, Oslo Lysverker contemplate three projects in the Aurland basin totalling 360 MW. The aggregate capacity in this area will thus be about 1,150 MW, and its output will be equivalent to about one quarter of the whole of Norway's present hydro-electric production.

Hol 1

Hydraulically, Hol I, when completed, will be virtually two stations in one. It will contain four machines, two of which will be supplied by the headwaters of the Votna under a gross head of 407 m. and the other two by the headwaters of the Hol under a gross head of 376 m. In addition, a main high-level storage has been provided which can supply either one or both of these systems according to the prevailing conditions of storage, and to obtain still further flexibility one of the Votna machines can be cross-connected to run at reduced head on Hol water.

The first of the Votna machines was put into operation in January, 1949, and the second in the following October, but the station ran at reduced load because the transmission voltage was held at 132 kV pending the completion of a new substation at Sogn, near Oslo. This eventuated in December, 1949, when the voltage was raised to its permanent value of 220 kV and the machines placed on full load. The main high-level storage has also been completed, but the Hol side of the scheme is still under construction and is due for completion in 1956.

The hydraulic system can best be understood by the help of the diagram given in Fig. 4, reference being made as required to the map, Fig. 2, for the locations of the various reservoirs and tunnels.

On the Votna side the immediate supply to the station is taken from Varaldsetvann, which is connected to the surge shaft by a tunnel 4,520 m. long by 18 sq. m. in section, and is impounded to raise the level from 999 to 1,005 m. The discharge at full load is 27 cu. m. per sec. This lake is itself fed through a 3 km. by 13 sq. m. tunnel from lake Rödungen, which has been raised from 1,011 to 1,022 m., and lake Rödungen in turn is supplemented from Bergsjö, impounded to raise the level from 1,079 to 1,081·5 m. and feeding through a short tunnel and open canal. There is also a natural flow to Rödungen from Tvistvann, lying at 1,060 m.

On the Hol side the main storage will be held at Strandevann, which is in course of being impounded to raise the level from 950 to 974 m., when the live storage will be 460 million cu. m. Later it is intended to raise the level a further 4 m. to 978 m., raising the storage to 554 million cu. m., and to drive tunnels to bring headwaters of the Aurland through the watershed on the west and headwaters of the Hivju from lake Halletjern on the east.

From the Strandevann intake a tunnel 7·65 km. long by 22 sq. m. cross section leads to a junction with a side-intake tunnel in the valley of the Urunda; thence the tunnel is enlarged to a section of 24 sq. m. and extends 9·65 km. to a surge shaft, known as the Urunda shaft, above Ruud station immediately to the west of the Votna surge shaft.

The high-level storage has been formed from a series of lakes forming the headwaters of the Urunda together with certain lakes that formerly fed the Votna. As already mentioned, the major lakes supplying the Urunda are Djupvann, Stolsvann and Olsennvann, the main lakes impounded from the Votna catchment being Buvann and Mjåvann. A number of channels have been cut between lakes where necessary and dams have been built on Stolsvann, Olsennvann and Mjåvann to convert a series of 16 lakes into one big lake having a total area of 35·8 sq. km. at an elevation of 1,091 m.

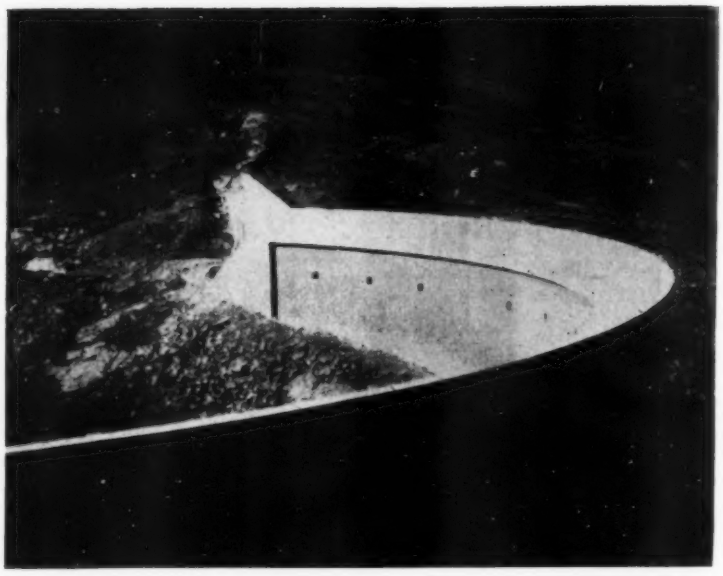

Fig. 7. Single-arch dam at Rödungen in the Votna catchment

Fig. 8. Varaldsetvann dam in the Votna catchment

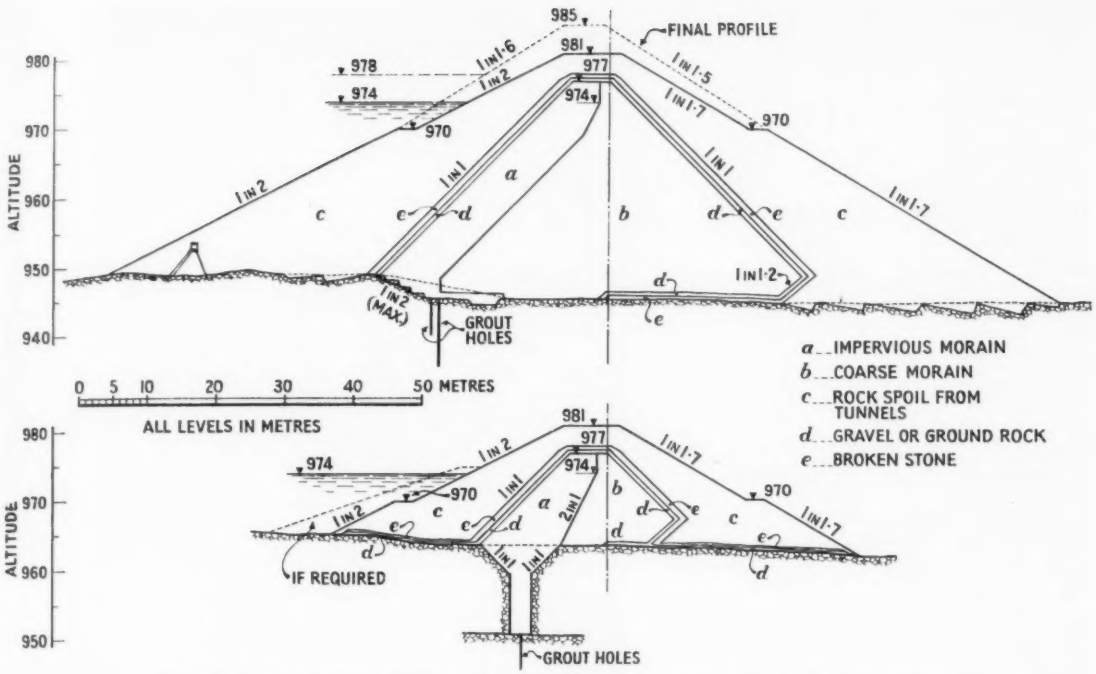

Fig. 9. Sections through the earth dam under construction at Strandevann

This storage can be drawn upon down to a level of 1,085·6 m. to feed either the Votna system or the Urunda intake as desired. On the Votna side water is released over the Mjåvann dam whence it flows to lake Rödungen. Water for the Urunda intake is released over the Stolsvann dam to flow down the bed of the Urunda to a weir adjacent to the portal of the side-intake tunnel mentioned above. Below 1,085·6 m. the supplies become separated, water being available to feed the Votna system down to 1,081·6 m. and to the Urunda intake down to a level of 1,078 m. This high-level reservoir has a total live storage of 219 million cu. m., of which 153 million cu. m. are available to the Votna.

Particulars of the major lakes impounded in the Hol-Votna catchment are given in Table I.

Dams

No less than nine dams are required for this scheme, and of these all except that at Strandevann have been completed.

A form of dam construction that appears to be popular in Norway is the reinforced-concrete flat-slab buttress type, known in that country as a plate dam. This is exemplified, fully or partially, in five of the dams under consideration, and was envisaged for part of the Strandevann dam until it was decided to adopt earthfill construction.

In the high-level storage the largest dam is at Stolsvann (Fig. 5), and is 18 m. high and 740 m. long. Of this length, the centre portion of 520 m. is of multiple-arch construction and comprises 13 arches each of 40 m. span. On each side is a plate-and-buttress section, and an open spillway. The two other dams controlling this storage are of the plate type; the larger is at Olsennvann (Fig. 6), being 11 m. high and 390 m. long, the other being at Mjåvann.

On the Votna system the Bergsjö dam is of the hollow type, 4 m. high, with a spillway over its full length of 100 m. Lake Rödungen has two dams; the larger is depicted in Fig. 7 and is a single-arch structure 13 m. high by 88 m. span, the other being a plate dam. Plate construction is also used at Veraldsetvann, the dam being 8 m. high and 88 m. long (Fig. 8).

On the Hol side one of the two largest dams of the development is under construction at Strandevann (Fig. 1). Originally this was conceived as a single-arch dam with a plate section on one side and a smaller single arch on the other, but later a decision was made to adopt earthfill construction. The dam will be about 40 m. high and 350 m. long, will contain 170,000 cu. m. of earthfill and 230,000 cu.

TABLE I MAJOR LAKES IMPOUNDED IN THE HOL-VOTNA CATCHMENT

System	Lake	Natural Level	Regulated Levels	Live Storage
		m.	m.	mill.cu.m.
Votna	Varaldsetvann	999	997—1,005	6·5
	Rödungen	1,011	999—1,022	79·3
	Bergsjö	1,079	1,070—1,081·5	12·8
Hol	Strandevann	950	950—974 later — 950—978	460·0 550·0
High-Level Storage	Mjåvann	1,084	1.081·6—1,091·0	219·0 of which 153·0 is available to Votna
	Buvann	1,089	1.087·4—1,091·0	
	Stolsvann	1,079	1.078·0—1,091·0	
	Djupvann	1,085	1,080·0—1,091·0	
	Olsennvann	1,084	1,083·0—1,091·0	

m. of rock, and will be the largest earth-fill dam in Norway. As will be seen from the sections given in Fig. 9, it will have an impervious core of local moraine towards the water side backed by a semi-pervious earth core, this earthfill being flanked by filter layers, outside which the dam is completed to section by rock spoil from the tunnels. Rockfill will also be used for the top of the dam, which is designed so that it can be raised 4 m. when required without disturbing the rest of the dam. A grout curtain is being put down in the rock below the impervious part of the core by grout holes 5 and 10 m. long.

Towards the right bank, where the foundation rock is higher, a deep cut-off trench has been cut in the rock, and 5 m. grout holes cut in the floor of the trench; as will be seen from the lower section in Fig. 9, the impervious earthfill will be carried down into this trench.

A spillway tunnel has been driven at some distance to the left of the dam.

When we visited the site a cofferdam had been thrown across the outflow to protect the dam workings, the foundation layers had been put down, and the cut-off trench in the rock abutment on the right bank had been cut and was being grouted. As will be seen from Fig. 9, the cofferdam will ultimately be submerged by the rockfill. Material was being handled by 13-ton Euclid trucks and compacted by tractor-drawn sheepsfoot rollers. Work was proceeding round the clock, laboratory control of the materials being exercised on all three shifts.

The only other dam on this side is the intake in the bed of the Urunda, which is 5·5 m. high and 85 m. long.

Fig. 10. Atlas drills with Sandvik Coromant steels on one of the tunnel faces

Fig. 11. Atlas-Copco loader mucking out in one of the tunnels

Tunnelling

The general tunnelling procedure is best exemplified by the methods adopted, and the results obtained, in the Strandevann-Urunda-Ruud tunnel, which is the longest in the scheme. This tunnel was driven on two faces in three shifts, employing six men on each face per shift. Including auxiliary workers the total labour strength was 61.

Drilling was carried out with light drills with airlegs and hard-metal bits, and a special technique was employed to obtain smoothly trimmed tunnel walls. Round the profile a series of holes was drilled at 40-50 cm. centres, and these holes were filled with half-charges of explosive alternating with spacers. This was done at the time that the rest of the face was charged, and the round was fired with the use of millisecond detonators. The resulting profile showed very little overbreak and was sufficiently smooth to impose relatively little boundary friction on the water although the tunnel was unlined.

Generally speaking, the rock was excellent both to drill and to blast, but near Urunda a stretch of 400 m. of quartzite was encountered which raised the drill-steel consumption from the usual 1¼ m. face advance per steel to 5 steels per metre. The normal weekly advance was 35-40 m. per face and the record was 51 m. The overall annual average was 1,600 m. per face per annum. At the beginning of the drive the average progress per man, including auxiliary workers, was equivalent to three hours per cubic metre of solid rock, but towards the end of the contract this fell to two hours per cubic metre, the best result being 1·5 hours per cubic metre. Taking faceworkers only into account, these figures are halved.

(To be continued)

Braithwaite Structural Pressed Steel. Braithwaite & Co. Structural Limited have published a particularly impressive brochure, in stiff covers, to call attention to their manufactures in pressed steel. Pressed steel tanks of all sizes and capacities are illustrated and described and, of special interest to the hydro-electric engineer, there is a short account of an elevated culvert, built up from standard components, to supply a water turbine.

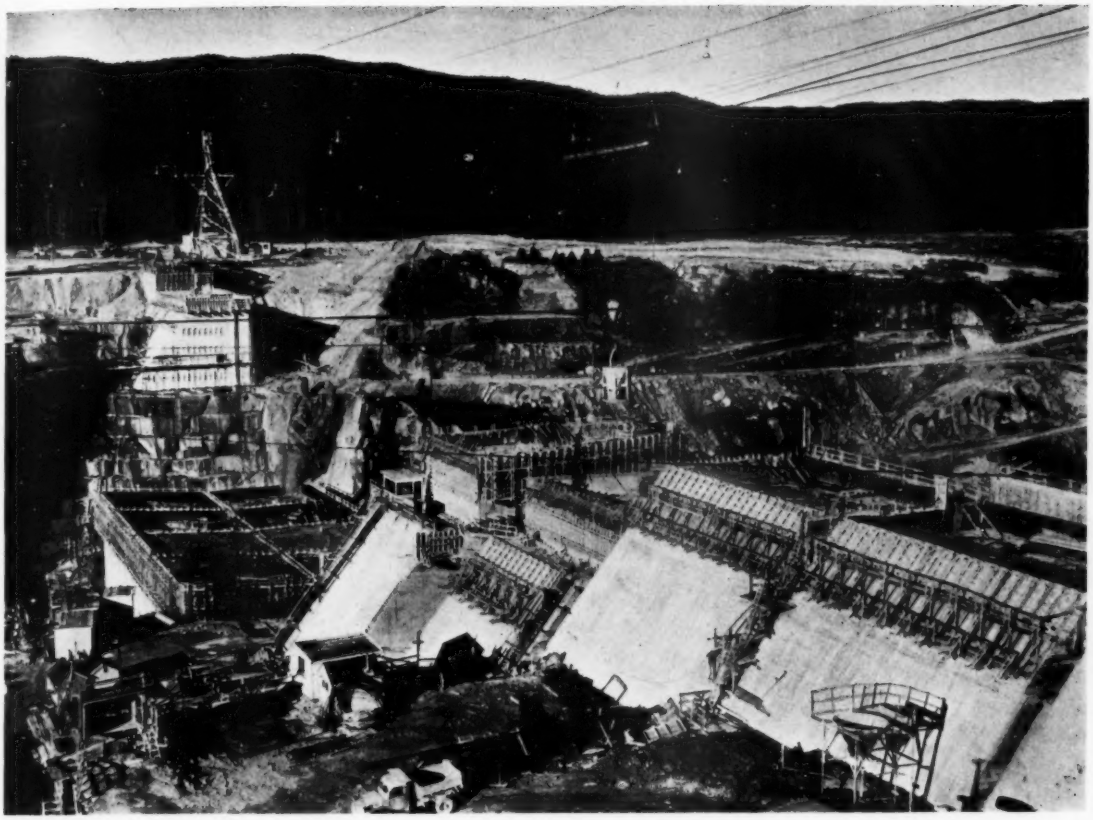

Fig. 1. Whakamaru power scheme showing main dam from cable-way operator's tower

New Zealand's Whakamaru Scheme

Brief particulars are given of this North Island scheme, the civil-engineering part of which has been designed and is at present being constructed by the Ministry of Works for the State Hydro-Electric Department.

WHAKAMARU is one of a succession of hydro-electric stations which will ultimately harness the power potentialities of the Waikato River. It is sited about 4½ miles from Mangakino, and will ultimately contribute 100 MW capacity to the North Island network, which is experiencing great difficulty in meeting the ever-expanding load, particularly in regard to such growing industries as pulp and paper production, the manufacture of fertiliser, and railway electrification. The central concrete gravity section of the dam contains the penstock intakes and the spillway. On the right bank there is an earth cut-off and on the left bank the concrete dam is flanked by an earth dam and earth cut-off. The power house is sited across the gorge immediately downstream of the concrete dam. The concrete section, which is reinforced with 452 tons of steel, has a crest length of 1,107 ft. and attains a maximum height of 183 ft. Sectional dimensions are maximum width, 154 ft., and crest width, 36 ft., the downstream face being sloped at 50° to the horizontal. A total of 275,000 cu. yards of concrete will be required for this structure; for the main hearting mix with a water/cement ratio of 0·65 the aggregate has a maximum size of 6 in. and the minimum cement content used has been 260 lb. per yard.

The earth-fill portion has a maximum height of 90 ft. at which point it has a basal width of approximately 500 ft. At the crest the width is 30 ft. and a roadway is carried along it which also extends along the concrete portion. A slope of 3 : 1 is maintained upstream and 2¾ : 1 downstream. Some 140,000 cu. yards of material had to be excavated to reach bedrock, and the dam will contain about 290,000 cu. yards of rolled earthfill when completed. The length

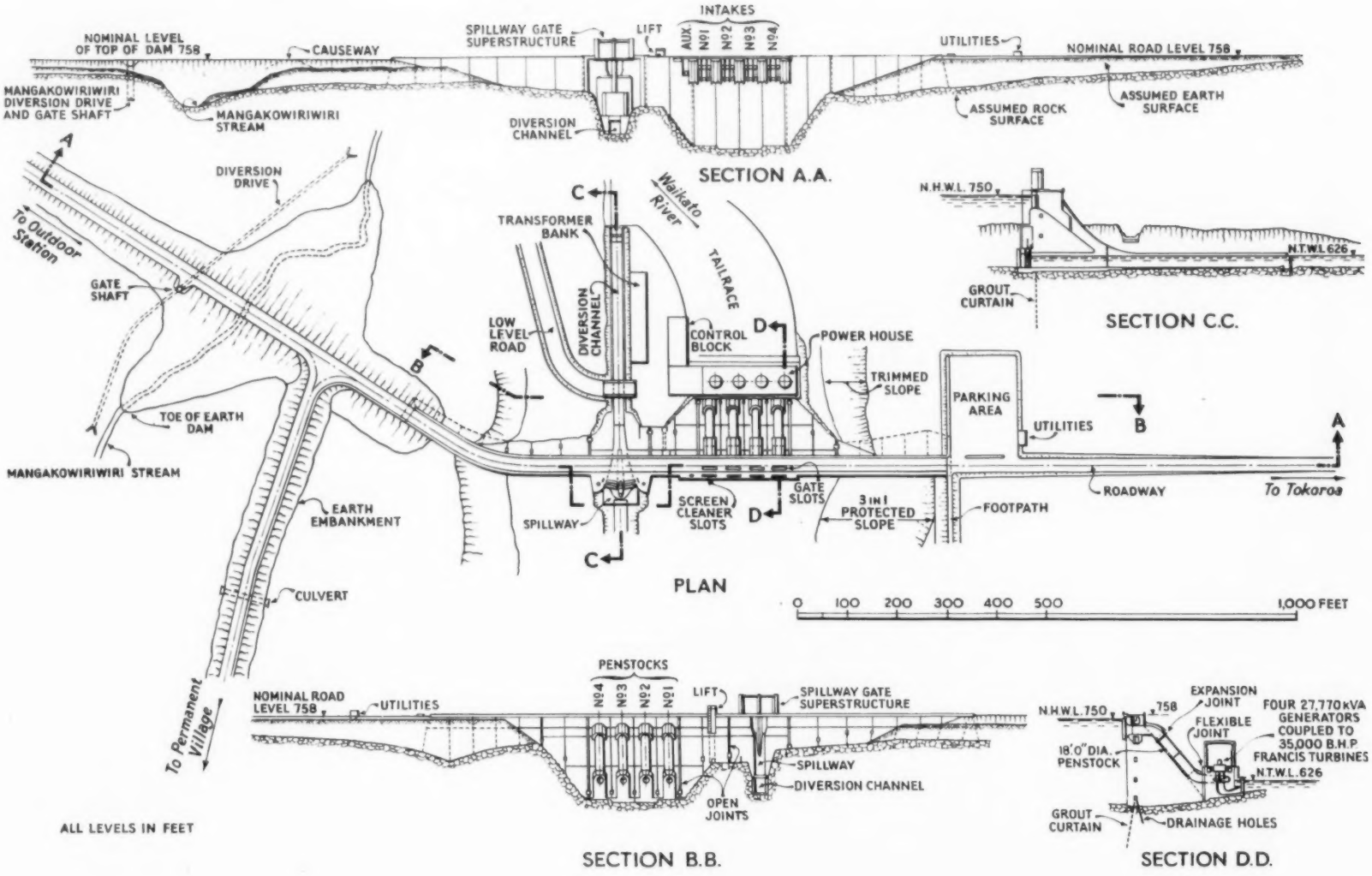

Fig. 2. Elevation and plant of Whakamaru dam and power station, with various cross sections

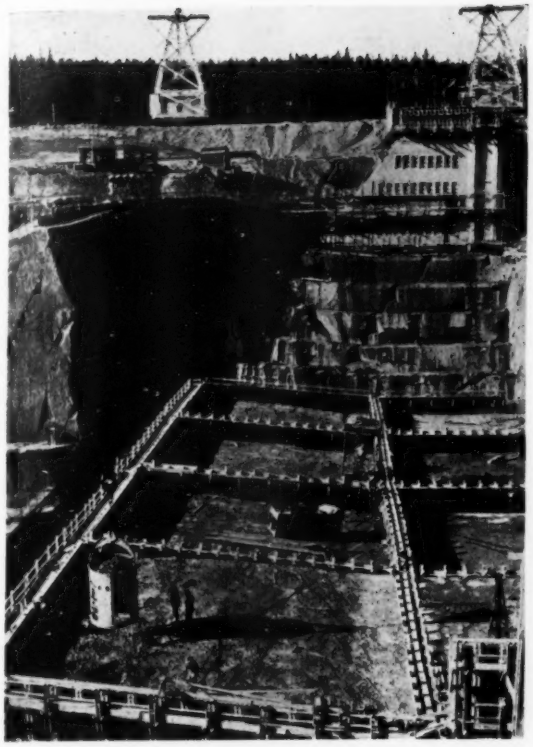

Fig. 3. Whakamaru dam, showing cableway delivering 3 cu. yard load of concrete to central portion

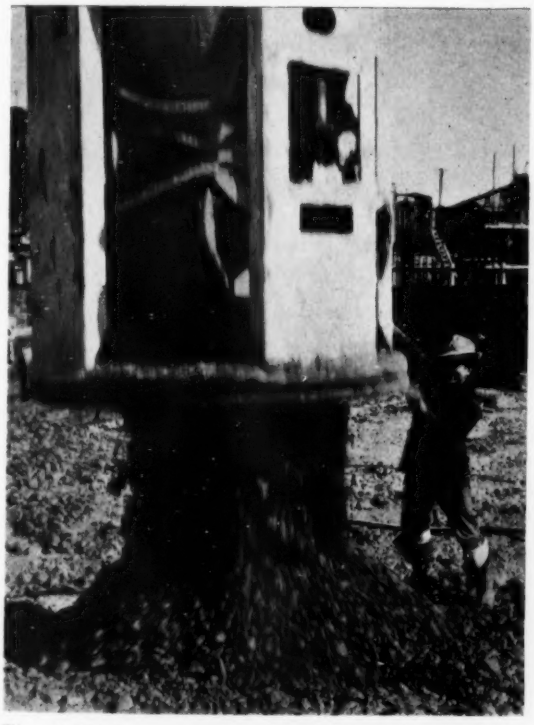

Fig. 4. Three cu. yard batch being unloaded from cableway bucket

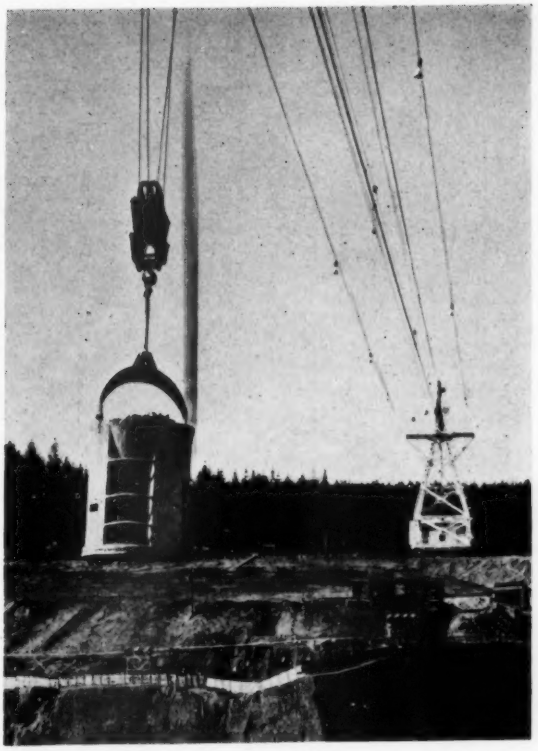

Fig. 5. The "Flying Fox" at Whakamaru; the bucket is of 3 cu. yards capacity

of cut-off on the right bank is 500 ft. while that on the left bank is 550 ft.

This barrier will impound 64,280 acre-feet of water contained in a lake which will have a length of 14 miles and a surface area of 3·1 sq. miles. A normal gross head of 124 ft. will be provided at the mean surface-level elevation of 750 ft. Normally the maximum and minimum levels will not range more than 1 ft. either above or below this level but in exceptional circumstances the minimum will be allowed to drop 2 ft., i.e. to an elevation of 748 ft. When the water level reaches E1. 752 the spillway will be capable of dealing with a flood discharge of 27,000 cusecs. The mean tailwater level is 626 ft.

To carry the river flow during the constructional period an open cut was excavated, 850 ft. in length and involving the removal of 122,000 cu. yards of ignimbrite rock—a fused volcanic tuff. This was carried to a maximum depth of 100 ft. below ground level and contained a culvert section measuring 23 ft. 4 in. by 27 ft. 6 in. The elevation of the invert section at the highest point is 605·5 ft. and at the outlet end 604 ft. The crest level of the upstream cofferdam has an elevation of 650 ft. while that of the downstream cofferdam is at 632 ft. With a flow of 6,000 cusecs during the diversion period the velocity was 13·5 ft. per sec. but when the scheme is complete and the spillway section of the dam is discharging at a rate of 27,000 cusecs a maximum velocity in the culvert will be 66·5 ft. per sec. The diversion sluice gate measures 23 ft. 4 in. wide by 28 ft. 6 in. high and is of the Stoney roller-face sealing type and weighs 80 tons. A direct-acting hydraulic cylinder, operating at

1,000 lb. per sq. in. maximum pressure, will operate the gate. This cylinder has a stroke of 29 ft. 3 in., a bore of 21¼ in., and has a lifting capacity of 147 tons.

The power house has an overall length of 264 ft. 6 in., the generator and lower turbine floors being at elevations 654·66 ft. and 630 ft. respectively. It is 60 ft. wide at generator level, below which it consists of a reinforced-concrete sub-structure while the structure above consists of a structural-steel frame, reinforced-concrete walls and a roof of precast concrete slabs covered with a layer of Neuchatel Bitumin ¾ in. thick.

The four penstocks have an internal diameter of 18 ft. and have a plate thickness of ⅝ in. at the top and ¾ in. at the bottom. Each is provided with an expansion joint at the upper end and a flexible joint at the lower end. They have a length of 216 ft. from intake to scrollcase and each carries 2,750 cusecs at the average velocity of 10·8 ft. per sec. The intake gates are spaced at 47 ft. centres and measure 24 ft. by 18 ft.; they are protected by bar screens having 3 in. by ½ in. bars spaced 4 in. apart. Each weighs 15 tons and is designed for a head of 35 ft.; they are of the Stoney roller type with rubber seals. Operating gear for each gate consists of a hydraulic cylinder which is capable of exerting a lifting force of 88 tons. The time for raising a gate is 3¾ min., excluding the penstock filling time, and for lowering, 20 sec. Two 4½ g.p.m. pumps, connected with a bus and having independent controls at each cylinder, provide pressure at 1,000 lb. per sq. in. maximum. Blakeborough bypass valves are provided, 30 in. diameter and having a penstock filling time of about 15 min.

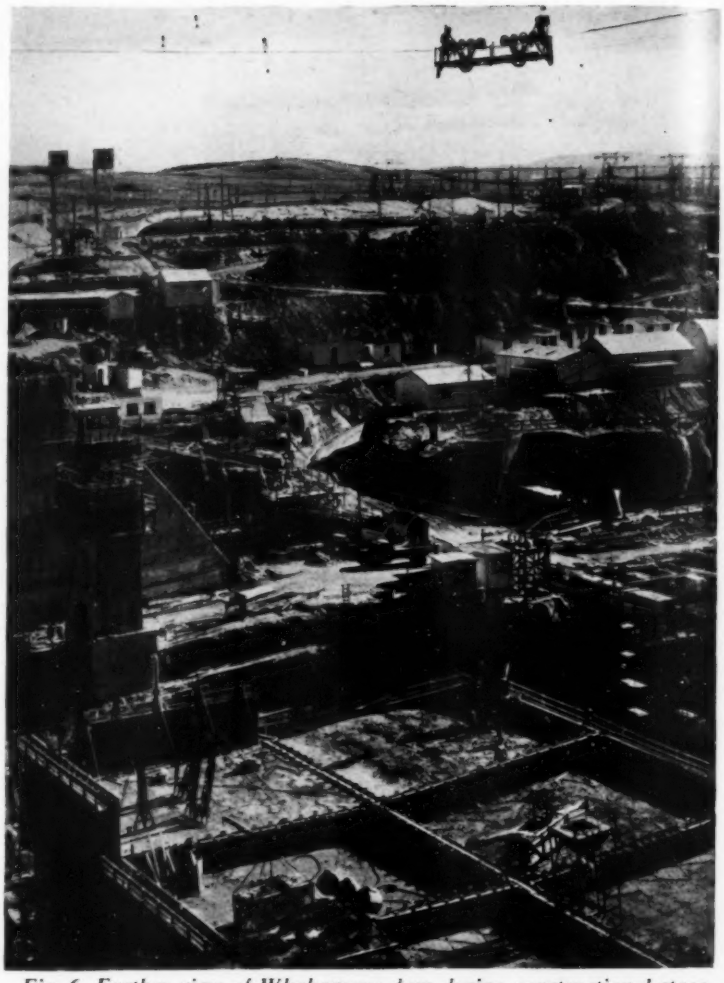

Fig. 6. Further view of Whakamaru dam during constructional stage

The spillway gates, 29 ft. high and 25 ft. wide, weigh 26 tons each and are provided with staunching rod seals. Lifting gear consists of two double-drum-type winches affording a lifting rate of about 1 ft. per min.

Stoplogs for the draft tubes measure 16 ft. by 12 ft., and weigh 5 tons each. They are designed for a maximum head of 30 ft. and are operated by a 10 ton capacity crane.

The generators are spaced 47 ft. apart and are serviced by two 80 ton capacity travelling cranes supplied by Cowans, Sheldon & Co. Ltd., Carlisle, England. The gantry rails are F.B. section weighing 112 lb. per yard. A traversing speed of 90 ft. per min. is achieved along the length of the machine hall and a transverse speed of 30 ft. per min.

All four turbines are of the vertical Francis type, manufactured by the Dominion Engineering Co. Ltd., of Canada, and are rated at 35,000 h.p. when operating on a head of 123 ft. At full load the efficiency is 90·3 per cent., rising to 92·2 per cent. at ¾ load. Each runner, which has a speed of 136·3 r.p.m., is 11 ft. 8 in. in diameter and has a weight of 44,000 lb. The total flywheel effect of the turbine and shaft is 600,000 lb. ft.². A maximum runaway speed of not more than 255 r.p.m. has been guaranteed. The generators, supplied by Metropolitan-Vickers Electrical Co. Ltd., of England, are rated at 27,777 kVA (25,000 kW, 0·9 power factor) and generate at 11 kV. At full load the efficiency is 97·69 per cent. and at ¾ load, 97·36 per cent. The rotating parts of the generator and exciter weigh 322,000 lb. and have a flywheel effect of 14 million lb. ft.².

The load on the thrust bearing due to the generator parts is 340,000 lb. while the total load amounts to 610,000 lb. Cooling water for the air cooler is circulated at the rate of 400 g.p.m. and the total required for each generator is 480 g.p.m., the pressure being kept below 25 lb. per sq. in. The breaking time for the rotor from full speed is about 2 min.

There are four banks of transformers, each having a capacity of 27,777 kVA and stepping up from 11,000 to 220,000 V. A spare single unit is also provided, the whole having been supplied by Ferranti Limited of

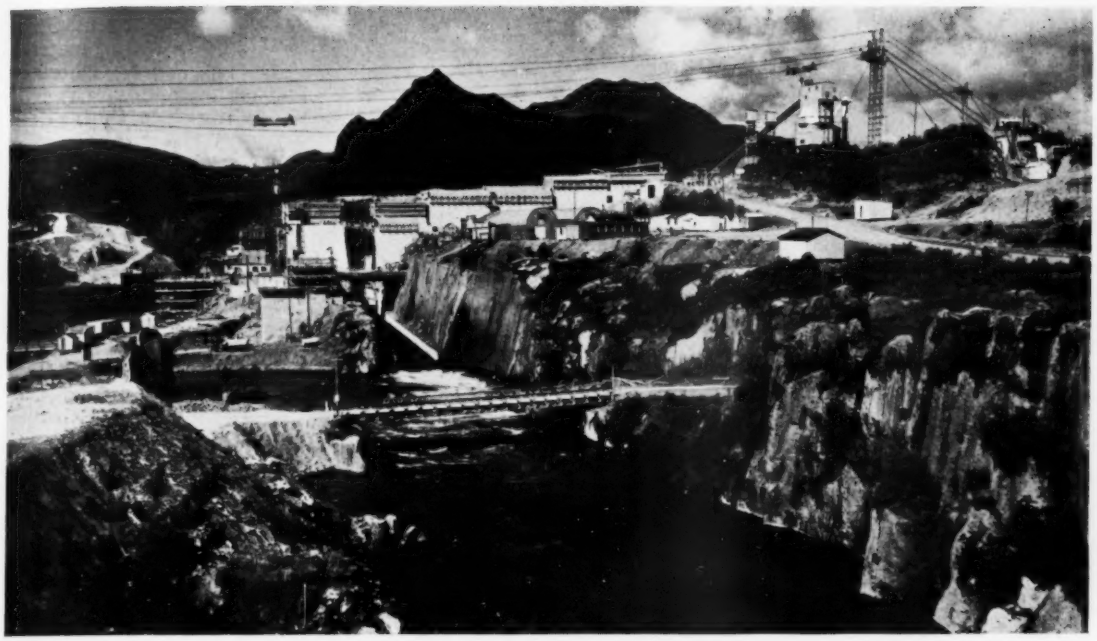

Fig. 7. View showing main dam, diversion channel through which the Waikato river (foreground) has been caused to flow, aerial ropeway, and (on right) concrete batching plant

Fig. 8. General view of operations at Whakamaru; the power station is being erected in the right foreground

England. These units operate on water-cooled forced-oil circulation, the cooling water required per single-phase unit being 110 g.p.m. at 25°C.

General

Constructional equipment includes two cableways each having a span of 1,200 ft. and a capacity of 7½ tons. The headmasts, 120 ft. high, are fixed while the tailmasts, which are mounted on a radial track, are 75 ft. high. The tail towers have a traversing speed of 60 ft. per min., while the travelling speed of the load is 900 ft. per min. A hoisting speed of 250 ft. per min. is achieved.

The main batching plant* consists of two 3 cu. yard capacity mixers and the aggregate, which is separated into six sizes, is kept in six bins of 340 cu. yard capacity. The cement bins contain 60 tons. Other equipment consists of two 3 cu. yard capacity material weighers, one cement weigher, one water weigher and one dispenser for an air entraining agent. The final aggregate is screened by means of two double-deck vibrating units measuring 10 ft. by 4 ft. Provision is made for adding Pozzolan if this procedure should be found desirable.

A cement silo of 1,000 tons capacity has been erected alongside the main batching plant and is fed by a conveyor and elevator from the debagging shed.

For permission to publish these notes we are indebted to the High Commissioner for New Zealand, who was also so good as to provide us with the photographs from which our illustrations were prepared. We also have pleasure in acknowledging the kind offices of Mr. C. W. O. Turner, Engineer-in-Chief of the New Zealand Ministry of Works, in supplying the data on which our notes have been based.

* This plant, manufactured by Winget Limited, England, was described in our May, 1952 issue, page 196.

Electrical Equipment for Kenya

An important contract for electrical plant at Juja Road substation, Kenya, has been awarded to the British Thomson-Houston Company. This substation has been specially designed in conjunction with the B.T.H. hydro-electric plant for Owen Falls generating station of the Uganda Electricity Board, and will transmit 40 MW of power to the Nairobi area of Kenya. The distance from Owen Falls to Nairobi is 330 miles and the power will be carried by the Uganda Electricity Board's 132 kV system to Tororo and thence by the Kenya Power Company's twin-circuit 132 kV transmission line to Nairobi. The overall design of the plant to be installed at the Nairobi receiving end will allow for the future possibility of the line from Tororo being tapped to provide a supply of electricity to the Eldoret-Kisumu area and the district surrounding Nakuru.

Balfour Beatty & Co. Ltd. are designing the receiving station at Juja Road, and the order placed with B.T.H. comprises the 132 kV and 66 kV oil circuit-breakers with their associated control and protective equipment, synchronous condensers, power transformers, and auxiliary switchgear. To provide for the control of power in the Nairobi area of Kenya and the local hydro-power stations at Wanji-Tana, the 66 kV switching station at Juja Road will become the focal point of this local system.

The 100th Smith Excavator for Canada

At a luncheon ceremony recently, Thomas Smith & Sons (Rodley) Limited celebrated the dispatch of their 100th excavator to Canada. We understand that they are the first British manufacturers to sell so many machines of 100 per cent. British design and manufacture to that important dollar market, and that the total value of these exports, including the necessary spare parts, amounts to one and a half million Canadian dollars. All this has been accomplished within the short space of five years, against very stiff U.S. competition, as well as that of other European manufacturers, particularly the Italians.

In the course of several visits to Canada, the directors, who have appointed agents in various centres from coast to coast, have been impressed by the need for flexibility in suiting their products to the Canadian market. The design of Smith excavators has accordingly been modified to meet the wishes of their Canadian customers, and these modifications have been found of general benefit to the firm's business in other overseas markets.

Dam Design Expert

John J. Hammond, Chief of the Bureau of Reclamation's Dams Branch in the Bureau's engineering offices in Denver, has recently retired after completing a 39-year career of distinguished Government service. He plans to remain in Denver, opening offices as an engineering consultant on the design of dams in America and abroad.

Mr. Hammond has supervised the design of virtually every major concrete dam erected by the Bureau, including Grand Coulee, Hoover, and Shasta Dams, the three highest and largest concrete dams in the world at the time they were built. Other notable dams designed under his direction include Seminoe, Kortes, and Glendo in Wyoming; Bartlett in Arizona; Parker in Arizona-California; Friant, Keswick, and Monticello in California; Marshall Ford in Texas; Thief Valley in Oregon; Altus in Oklahoma; and Hungry Horse and Canyon Ferry in Montana. He also directed designs of the Norris and Wheeler Dams for the Tennessee Valley Authority; Madden Dam for the Panama Canal Zone; and Tumut Pond and Junction Dams for the Snowy Mountains Hydro-Electric Authority of Australia. Additional major dams designed under his supervision include Anchor in Wyoming and Yellowtail in Montana, both of which were recently authorised for construction by the Congress.

Mr. Hammond has travelled abroad to advise the Snowy Mountains Hydro-Electric Authority on design of dams in New South Wales, and has also advised the Governments of India and Formosa. He was consulted on the designs for the New Cross Dam (Dam No. 22) in Colorado, built by the City of Denver Water Board.

Mr. Hammond is a member of the International Commission on Large Dams, the International Commission on Irrigation and Drainage, the American Society of Civil Engineers, and the Association for the Advancement of Science.

Tunnelling through Unstable Rock

A special method successfully used in Sweden is described by Prof. Dr.techn. L. RABCEWICZ and Ing. L. MONSEN

WHEN driving a tunnel in sound homogeneous rock it is generally most economic to blast it out full face unless the cross section is very large. If the area exceeds about 120 sq. m., however, it is usually more advantageous to divide the section into a lower and an upper part, driving the latter first. This method can still be used with advantage even if the rock conditions deteriorate, so long as the roof can be bolted. In any case it quite often happens that, after blasting, some rock will fall and leave an arched overbreak pocket which is either supported between the solid or between the consolidated rock of the last round and the face.

To attempt to secure such a pocket with net and roof bolts, without taking measures beforehand for the safety of the crew, may very well lead to accidents, especially if the tunnel is large and has, in consequence, a relatively small arch effect in the cross direction. Such safety measures may consist of underpinning the dangerous ground by a solidly built drill jumbo or using crown bars, the latter being carried on specially fabricated hangers secured to the bolts of the previous round. All these preparations will cost time and money, in addition to the expense of the additional bolting and netting which is always necessary in such a case.

The method still mostly practised in these conditions is to use steel arches and lagging and to pack the overbreak volume with wood. This is not only very expensive but is also undesirable if the wood packing is left in the concrete.

A new method has been tried in a discharge tunnel of 175 sq. m. sectional area at the Edenforsen power station, North Sweden, for several sections of bad ground. The geological conditions were examined very carefully beforehand by means of numerous borings which disclosed that there was very little rock overburden over a certain section and that the rock below was of inferior quality. It consisted, over a length of about 50 m., of weathered and shattered granite, with many clayfilled joints; furthermore, minor fault zones not revealed by the borings were considered to be probable. In order to be prepared for various kinds of support problems in the specially difficult portions of disintegrated rock likely to be encountered, it was decided to drive the upper part of the tunnel first, with a face area of 70 sq. m. The problem to be solved was to find a method which would support the rock, protect the crew and would not delay progress. To meet these demands a steel structure was designed by the writers, which might be termed a "movable protective shuttering," consisting of two very simple steel forms of polygonal cross section (Fig. 1), each 12 m. wide, 6 m. high and 2 m. deep and provided with hinged joints. The folding sections, which are folded up during transportation, can be fixed in various working positions in order to save concrete in places where there is excessive overbreak at the sides. The principal idea of this construction was to avoid the erection of any kind of temporary support by moving the form into the insecure zone as soon as mucking out and a rough scaling had been effected. Once placed to line and grade, bulkheading the shield and pouring the concrete could be carried out under protection of the form.

In order to increase the degree of safety a steel net should be placed in position and supported by concrete blocks which are wired to the form. Should a piece of rock fall down it then hits the resilient net which yields elastically to reduce the impact to a negligible amount. As a further safety measure it was intended to underpin the anglepoints by steel pipes, as shown in dotted lines in Fig. 1, but this precaution was proved to be unnecessary in the case in point.

Moving of the form, which weighs 10·3 tons, from one place to the other was effected by a 13 ton

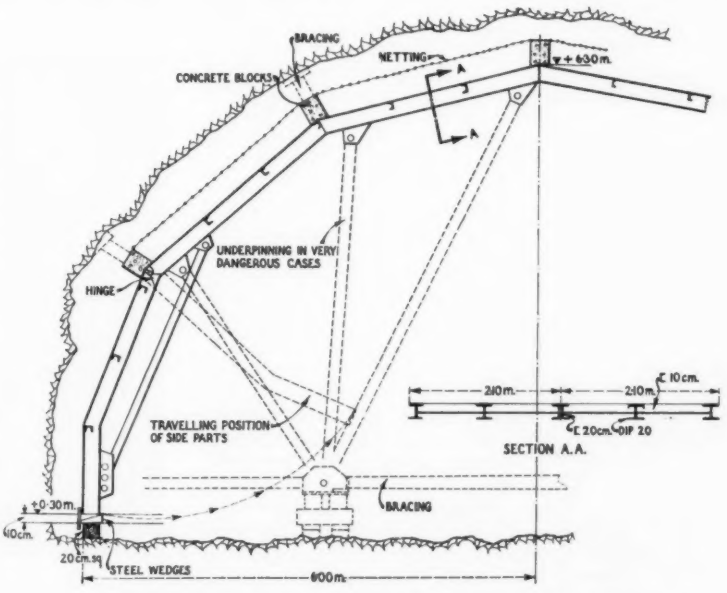

Fig. 1. General arrangement of protective shuttering

Fig. 2. Transport scaffolding being lowered on a "Mack" dumper

"Mack" dumper. A simple scaffolding was set up consisting of four braced steel pipes with a jack at the top of each, the jacks having a travel of 70 cm. (Figs. 2 and 3). The dumper was charged with a rock load of 5 tons to increase its stability.

To make the manner of working quite clear, a complete cycle will be described in detail: After allowing the necessary time for the concrete to harden, the dumper carrying the scaffolding is driven into position, the jacks extended until they take load, and the folding sides released from the concrete and winched in by hand. The form is then carefully lowered by screwing down the jacks, cleaned and oiled. If a protection of concrete blocks and net is needed it must be done at this stage of the work. Afterwards the form is moved forward either to the adjacent section or to a further weak place needing support.

After stopping at the new place the form is jacked up, the sideforms being winched out at the same time. Four to six men then lever the form in the desired direction with crowbars. Two 20/20 cm. wooden beams are then laid longitudinally underneath the sides of the movable form and 50/50 mm. steel wedges inserted. Thereafter the jacks are screwed down and the form lowered on to the wedges. While completing the form by closing the gaps between the wooden beams with planks, the dumper is driven away and the drill jumbo brought in for use as a working platform for erecting the bulkhead at the end of the form. At the same time the concrete placer is brought to the spot, the pipeline set up and the concreting started. After the concreting is finished, the next round is drilled, loaded and shot. Experience showed that the blasting of a round close to the newly concreted section, after only six to ten hours from the time that concreting was finished, did not do the least damage. To speed up the hardening process, the concrete had been given an admixture of 2 per cent. of calcium chloride. Finally, mucking is done and the cycle is closed, the form then being ready for use again. Fig.

Fig. 3. Protective form being lifted by transport scaffolding to be taken into tunnel

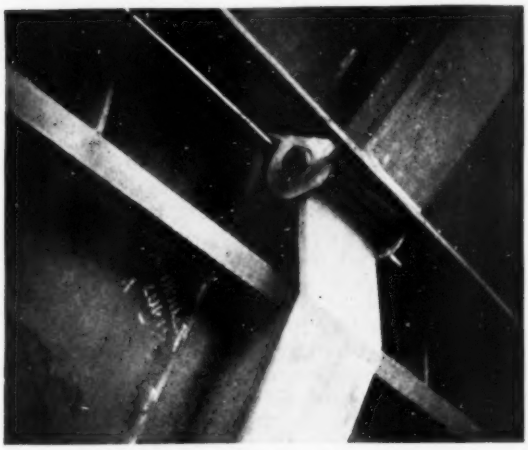

Fig. 4. Detail of hinge

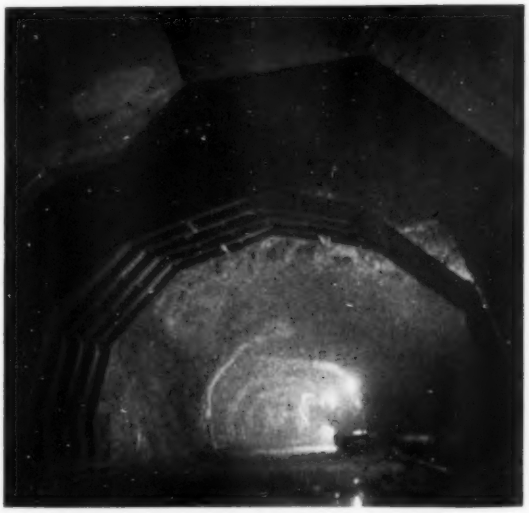

Fig. 5. Form in position ready for bulkhead to be erected; a concrete section can be seen in the foreground

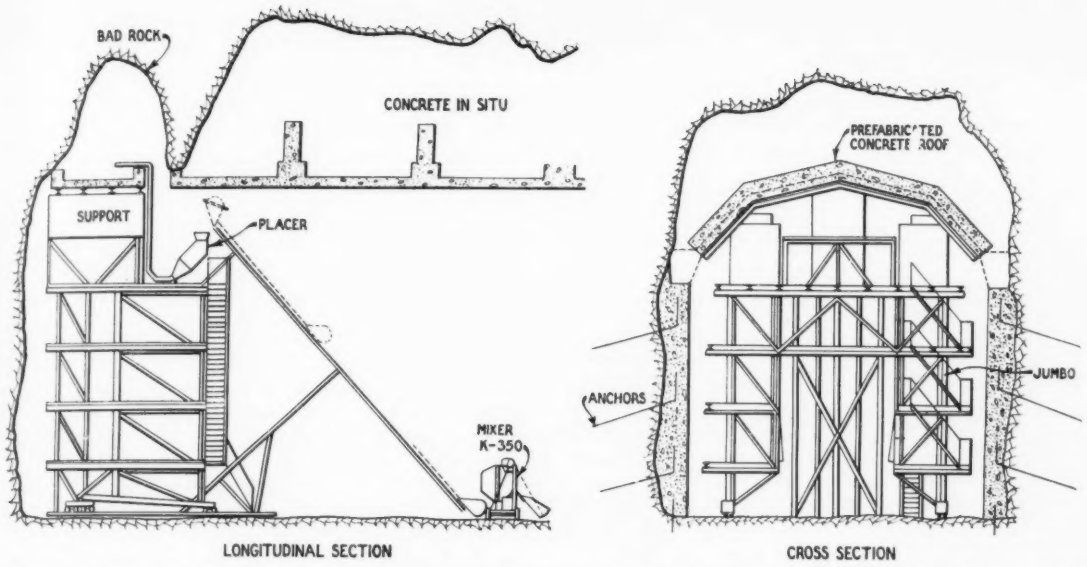

Fig. 6. Concreting the side walls and abutments under cover of a prefabricated concrete roof (discharge tunnel, Dönje water-power station, Sweden)

5 shows the form in an unstable section of the tunnel.

After a few cycles the crew got sufficiently acquainted with the work to perform it quite quickly. The different operations and the time of their execution over a 4 m. length are given in Table I.

TABLE I TIME TAKEN IN HOURS

1. Dismantling, transport, erection and bulkheading of the form	10
2. Concreting, including placing and taking off gun and pipes	18-22
3. Drilling, blasting and mucking	16
The whole cycle thus	44-48

The 10 hours mentioned in 1 does not include the time taken to erect the special protection afforded by the concrete blocks and netting. The men took the risk of going on without it and there were no unfortunate incidents.

Operation 1 was performed by a crew of six men in 10 hours. This corresponds to about 0·75 hours per sq. m. surface of formwork (bulkheading included) which is certainly a very good rate, illustrating the usefulness of the device. The time taken for operation 2 could be considerably shortened by further mechanising the concreting equipment.

The method is, of course, very cheap as the roof had to be concreted anyway and no temporary reinforcement of any kind was needed. The initial cost of the formwork (two sets of 2 m. length) was 13,000 Sw.kr. delivered at the site. To this must be added the cost of the transport scaffolding, 3,000 Sw.kr., making a total of 16,000 Sw.kr. (£1,100). This is equivalent to 220 Sw.kr. per sq. m. and 1·53 Sw.kr. per kg., respectively £15 and 2s.

If protection of concrete blocks and netting had been applied, the cost of delivery and placing of the net would have been the only additional expense.

There is naturally some loss of time with this method compared with roofbolting or the use of steel arches, but only if the concreting is done simultaneously with the driving. If conditions allow concreting to be started after the tunnel is holed through, the loss of time is relatively very small.

Taking into consideration the advantages and disadvantages of the "protective form" method, it can be accepted as being useful and economic for large tunnels which run mainly through good rock, which can be left unlined, but which contain patches of bad ground which need to be lined. Such geological conditions are fairly common in Sweden and often occur elsewhere. In Sweden it is considered preferable not to line a bad section as a whole—provided, of course, it is not too bad—but to use concrete arches of from 1 to 2 m. in length, at intervals, leaving the intermediate rock unlined. The "protective form method" is eminently suitable for such an intermittent method of lining. Naturally, the length of the arches must economically be made equal to the form.

Still another method has to be mentioned which was applied at the Dönje tunnel in Northern Sweden.* The principal idea of this method is similar to the one just described, and was applied to zones of shattered rock and clayfilled faults which repeatedly caused rockfalls, making it imperative to protect the crews. Instead of using temporary supports, a concreted section of the roof, several metres in depth, about 25 cm. thick, weighing 30 tons, was prefabricated on the jumbo; it was then carried forward into the weak zone by two tractors and jacked in the desired position. The abutments to support this roof were then shuttered and concreted under the protection of the concrete cover. The space between the rock and the roof segment was subsequently filled with concrete. The method proved to be effective (see cross-section in Fig. 6) but it is certainly slower and more expensive than the one described previously.

* Broms. "Sprängning av avloppstunneln för Dönje Kraftverk." Svenska Vattenkraftföreningens publikationer 1953/4.

The Geneva Conference

The first International Conference on the Peaceful Uses of Atomic Energy was held at the Palais de Nations, at Geneva, from August 8-20, 1955, and it was perhaps one of the most important scientific conferences ever to be held. There were 1,270 delegates and advisers, and perhaps the same number of observers, those attending being drawn from 72 countries.

By J. H. M. SYKES, Assoc.I.E.E.

IT would not be appropriate in the columns of this journal to endeavour to assess the results of the International Conference on the peaceful uses of atomic energy from the point of view of nuclear power generation, but two essential facts emerge. First, technology in the nuclear field has now progressed far enough for at least three countries—United States, Great Britain and the U.S.S.R.—to have embarked on major programmes of nuclear power generation, Great Britain being the most advanced in this respect, with the first commercial-scale station being planned for commissioning early in 1956. The Russian engineers have had a 5,000 kW station in operation satisfactorily since June, 1954, and it may be added that not the least important aspect of the Geneva Conference was the freedom with which engineers and scientists from both sides of the "Iron Curtain" exchanged technical information with very little reserve. Secondly, although on the economic side there was less certainty in regard to figures than on the technical side, it is fairly certain that nuclear power will be able to compete with power generated by the burning of coal in conventional stations, although not necessarily, for many years to come, with hydro power generated at stations where the capital cost has been kept to a minimum.

The first section of the conference was devoted to a series of papers on the world's energy needs over the next 50 years. One of these papers, by Dr. E. A. G. Robinson, of Cambridge University, and Mr. G. H. Daniel, of the Ministry of Fuel and Power in London, on "The World's Need for a New Source of Energy" had this to say about hydro power:

"Only a fraction of the theoretical total of the water power of the streams of the earth can in fact be harnessed. The best estimates are that the potential water power (including what has already been developed) is of the order of 550 million kW, measured at minimum flow. The amount of developed power at existing sites is usually two to four times, and may be as much as ten times, the potential power at low flow. A fair measure of the total potential amount of energy that could be produced may be got by taking three times the minimum flow potential and assuming that this is fully used for half the year. This yields an estimate of 7,000 milliard units a year, which could be produced at present-day average efficiencies by about 4 milliard tons of coal a year at thermal power stations, or about 2½ milliard tons at the efficiencies that may prevail after the year 2000. Since it is uneconomic to transmit electricity over long distances the full exploitation of this power will depend on the development of industry in isolated parts of the world—for instance, the Belgian and French Congo—where much of the potential is located. The tendency for reservoirs to silt up is a serious factor which must not be overlooked. The most optimistic assumption is that, in the absence of nuclear energy, the number of units generated would grow at the rate of 4 per cent. a year, so that by the year 2000 hydro power would be saving nearly 1 milliard tons of coal a year."

The authors go on to quote the recoverable world reserves of energy, as given below:

RECOVERABLE WORLD RESERVES OF ENERGY
(Milliard tons of coal equivalent per annum)

Coal and Lignite	3,000
Oil and gas	300
Oil shales and tar sands	150
Hydro-Electricity	2·5

A further table gives the authors' estimates of the possible demand for the conventional sources of energy in the absence of a new source.

POSSIBLE DEMAND FOR THE CONVENTIONAL SOURCES OF ENERGY IN THE ABSENCE OF A NEW SOURCE
(Milliard tons of coal equivalent per annum)

	Total Energy	Coal	Hydro-electricity	Oil and natural gas
1950	2·8	1·6	0·2	1·0
2000	7·4	2·0	0·9	4·5
2025	12·1	2·3	2·2	7·6

In dealing with the costs of the conventional sources of energy, the authors say, "Although the capital costs of hydro-electric stations are usually (though not always) higher than those of thermal stations of the same capacity and there are wide variations between individual plants, the cost per unit of hydro-electricity supplied today is on an average substantially below the cost of energy from thermal plants. At first sight the industry would seem to be subject to diminishing returns, as the most attractive sites are likely to be developed first. Various studies have shown, however, that improvements in engineering techniques have so far largely offset this tendency. Though technical progress may not continue to compensate for the natural disadvantages of the sites that must progressively be harnessed if the world's potential is to be exploited, it is likely that the cost of hydro-electricity will remain sufficiently below that of the power produced from coal-fired stations to justify continued rapid expansion of water-power reserves, until nuclear power stations offer a more attractive alternative."

Dealing with the last point raised by these authors, it is appropriate here to quote a document made avail-

able at the time of the conference, and written by Mr. Walker L. Cisler, the President of the Detroit Edison Company. Mr. Cisler says, "We have endeavoured to look into the future and to discern, even dimly, the stages in the emergence of atomic energy as a universal influence. It has seemed reasonable to expect:

1. In ten years—development of means for economic utilisation of nuclear power.
2. In 25 years—wider utilisation to provide a relatively small part of the total electric power used.
3. In 50 years—fuller utilisation to provide much of the new electric power requirements then existing."

Another approach to an estimation of the part played by water power in the world was given in a paper presented on behalf of the United Nations Statistical Department by Mr. N. B. Guyol. In this case, the unit adopted was different again, and the figures given below are in thousand million MWh electricity equivalent.

WORLD PRODUCTION OF FUEL AND POWER IN 1952
(In thousand million MWh electricity equivalent)

	Quantity	Percentage of total
Coal	12·0	41·4
Lignite and Peat	1·3	4·5
Petroleum and natural gasolene	7·7	26·5
Natural gas	2·7	9·3
All mineral fuels	23·7	81·7
Water power	0·4	1·4
Vegetable fuels	4·6	15·9
Animate energy	0·3	1·0
Total	29·0	100·0

The conclusion reached by this author is that while total requirements of energy may be three times as large in 1975 as they were in 1952, and eight times as large in the year 2000 as they were in 1952, fuel supplies will probably need to increase at a rate of only about 3½ per cent. per annum to keep pace with requirements, even at the high levels indicated. Should nuclear energy become economically available for the production of power and heat, the total fuel requirements might be of an entirely different order.

Coming hard on the heels of Mr. Guyol's paper from the United Nations Secretariat, there came another from the same source, prepared by Mr. P. Servette. He took an extremely realistic view of the estimation of world hydro-electric resources, when he said, "The precise evaluation of hydro-electric resources is very difficult. All that may, in fact, be determined with some certainty is their upper theoretical limits. A determination of these limits has been made the subject of recent studies based on a uniform method, to enable figures, which are comparable between themselves to be obtained. ("The Hydro-Electric Potential of Europe, and its Theoretical Technical and Economic Limits." U.N. document E/ECE/EP/131, May, 1953*.)

"But it is the knowledge of the resources which are technically and above all economically possible to harness which is the most difficult to come by . . . The technical and economic potentials usually represent about 30 and 20 per cent. respectively of the theoretical potential. These ratios, however, only give an order of magnitude, and the determination of the economic potential depends, according to the country concerned, on the number of advance studies which have been made, progress in technique, the degree of exploitation already accomplished, the energy needs of the country, and the presence and location of other sources of energy. Thus it is necessary for evaluations of hydro-electric potential to be made periodically. It is only necessary to cite one example to demonstrate this: In Sweden the calculations made in 1923 gave a potential of 32 milliard kWh, but this figure was raised to 40 milliard kWh in 1938, and the potential is now estimated at 80 milliard kWh.

"The annual development of this hydro-electric potential is primarily limited by the capital available and the long duration of the construction period applicable to practically every type of hydro-electric power station."

This paper is particularly valuable for a most detailed world assessment of hydro-electric and other power resources. Some figures from this table which may be of interest are set out below.

	Hydro-electric energy reserves in milliards of kWh per annum
North America	442
Central America	95
South America	368
North-East Africa	48
Central Africa	1,481
South Africa	2
U.S.S.R.	464
North-West Europe	363
East Europe	45
Mediterranean Europe	166
West Asia	96
South Asia	211
South-East Asia	100
Far East (China, Japan, etc.)	821
Oceania (Australia, New Zealand, Pacific Islands, etc.	47
World Total	4,749

The President of the Conference on Atomic Energy, Dr. Homi J. Bhabha, said that "In a broad view of human history it is possible to discern three great epochs. The first was marked by the emergence of the early civilisations in the valleys of the Euphrates, the Indus and the Nile; the second by the industrial revolution, leading to the civilisation in which we live, and the third by the discovery of atomic energy and the dawn of the atomic age, which we are just entering."

Whatever may happen to the other conventional sources of power, there is no doubt that in those countries fortunate to possess hydro-electric resources, water power will be the permanent partner of the atom in this new era.

The Riveting of Aluminium. The Aluminium Development Association have issued information bulletin No. 8 (price 2s. 6d.). This revises the first edition which dealt with rivets up to ⅜ in. in diameter, and the new book outlines the successful development of aluminium rivets up to 1 in. shank diameter.

Reyrolle Equipment. Pamphlet No. 1281/4-55, issued by A. Reyrolle & Co. Ltd., of Hebburn, describes this firm's Type-JKSS oil-immersed ring-main switch and tee-off fuse-switch unit for outdoor service. It is essentially non-extensible and is specifically intended for situations where only a single tee-off fuse-switch is required; it is built for service at up to 11 kV 250 MVA.

* See WATER POWER, August 1953, pp. 281 and 284.

The Study of Dam Sites

This subject is discussed in relation to the depth at which good foundations are to be found, the strength of those foundations, the chance of leakage through them and any special features of the rock which are likely to have a bearing on the successful erection of a dam.

By DR. J. E. RICHEY, M.C., F.R.S., F.R.S.E.
Consulting Geologist, Edinburgh

A WELL-KNOWN Swiss geologist, Maurice Lugeon, once made the remark[1]: "Elever un barrage, c'est faire une expérience"—to erect a dam is to make an experiment. It is never certain what geological features will be discovered when a dam site is excavated, however much preliminary investigation and thought may have been given to the problem. Nor, perhaps, are the engineers ever free from doubt about every item in their construction until after a dam is completed and the reservoir filled. Even years later, unforeseen and unpredictable weaknesses may appear. In the United States, for example, alkali reaction in concrete affected a number of dams and other structures many years after they were built. The

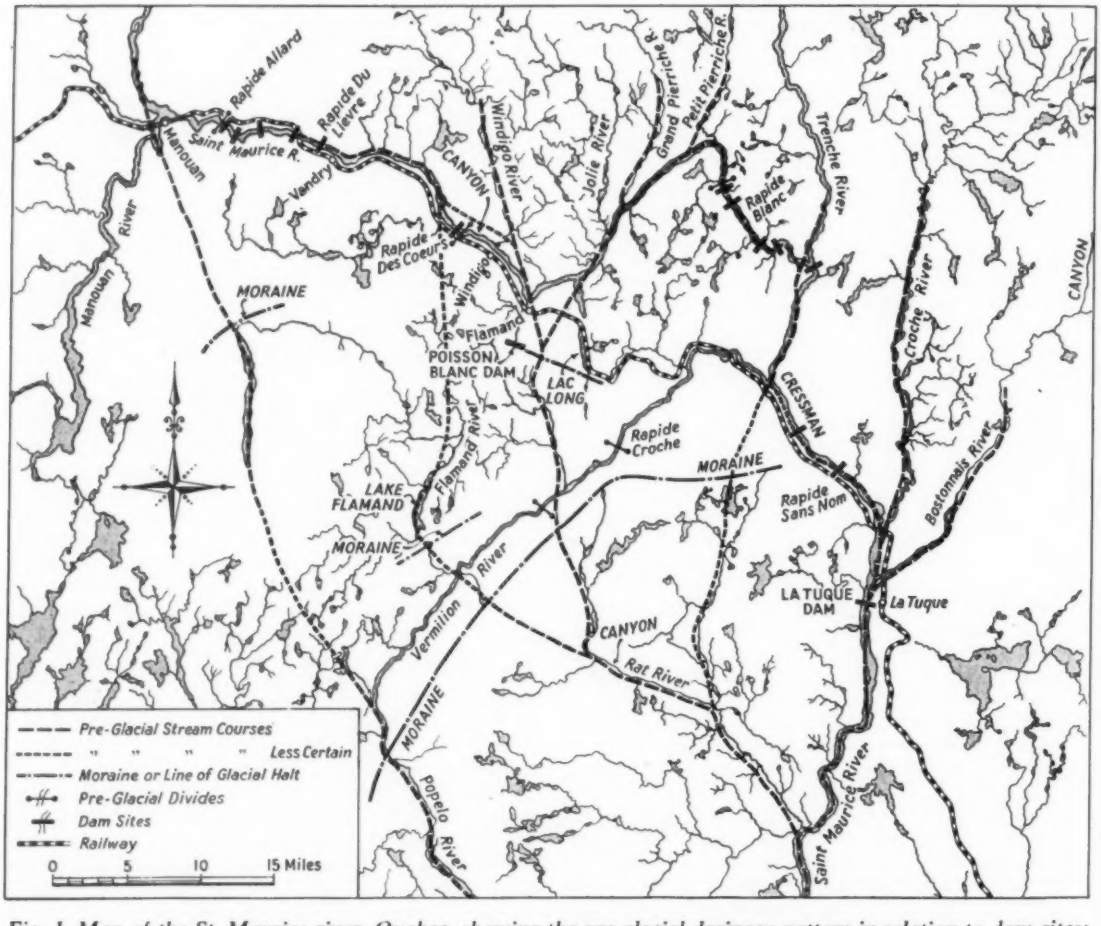

Fig. 1. Map of the St. Maurice river, Quebec, showing the pre-glacial drainage pattern in relation to dam sites: likely sites occur where a river crosses ancient rock water sheds. (After Irving B. Crosby, part of Fig. 1., p. 336, Boston Society C.E. 23 (1941))

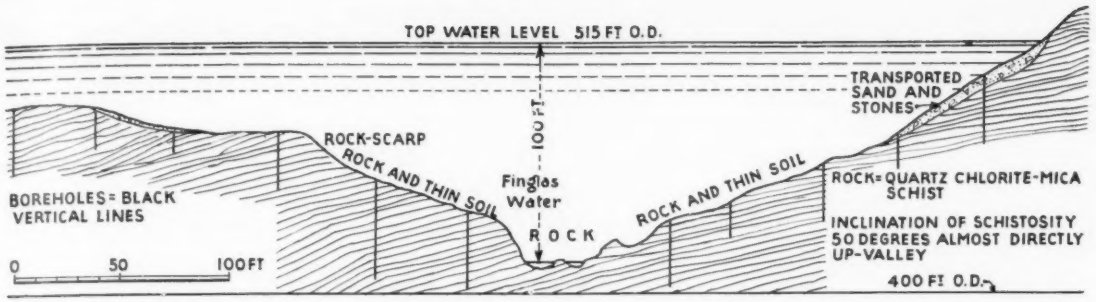

Fig. 2. Example of a recently eroded valley with sound rock well exposed; section of the dam site across Glen Finglas, east of Loch Katrine, Scotland. (By permission of Stanley D. Canvin, Chief Engineer, Water Department, Corporation of Glasgow)

cause was discovered by T. E. Stanton in 1940 and, as is now well known, two materials were found to be involved, namely, high alkali cement and an adequately reactive aggregate, mainly opal or opaline silica. It is, however, solely with geology and its materials that most unanticipated difficulties have lain, and not with cement or with engineering design and workmanship.

Every site has some geological peculiarity or other, but the chief information that the engineer requires from the geologist may be stated in general terms. This concerns questions relating to the depth at which adequate foundations exist, the strength of these foundations, the chances of leakage through them, and any special features of rock or "soil" which, as far as can be seen, are likely to have any bearing on the excavation work itself.

Depth of Excavation

Three geological situations may be encountered, and all three are frequently present at a single site. They are: (1) Rock exposed at the surface, the rock being in good condition and little weathered. (2) Rock decomposed and covered with residual soil. (3) Rock covered with transported "soils," or "drift" deposits. Three geological criteria are involved in these matters —the history of the river, the recent climatic conditions in the region, and the degree of durability of the rocks. Each of these forms a subject in itself, and it is possible here to refer only to a few of the more outstanding points.

The River

All rocks, however durable they may be, fail and disappear in course of time owing to weathering and denudation that is constantly taking place. Knowledge of a river's history will supply some idea of the time during which the gap to be dammed has been in existence, and so give some measure of the extent to which the rock concerned may have undergone decomposition. For example, during the recent Ice Age river-action over most of the British Isles was discontinued. On the disappearance of the ice-sheets rivers in many places followed new courses, and the time interval, geologically speaking, for the new channels to be formed has been relatively short—only 10,000 years or so in the mountains. Again, recent earth movements, such as the uplift of a region, bring about rejuvenation of river action; that is, where a river was formerly depositing sediment along the floor of a valley, it may now be eroding its bed. Consequently, sound rock may be expected to occur, at most, at only a slight depth below the stones and boulders forming the valley floor. Active erosion, however, may lead to a different situation. Where a river bed is obscured by stones and boulders, it may be that the slow traction of these materials along the rock-floor of the river has eroded and over-deepened the channel. A suggestive case was encountered at the Laggan dam-site in Inverness-shire, Scotland, where a dam 130 ft. high was constructed in 1935 or thereabouts for impounding water for the British Aluminium Company's works at Fort William. When excavated, the rock-bed of the river was found to be covered to a depth of 35 ft. with large and small boulders. The coarse nature of the deposit had prevented exploration by boring[2]. Probably the most impressive example on record of the over-deepening of a river-channel by traction was likewise not discovered until excavation began, this being at the site of the Boulder (Hoover) Dam on the Black Canyon of the Colorado River. As is well known, an inner gorge filled with sand, gravel and boulders was found to have been eroded within the bottom of the gorge to a depth of 75 to 80 ft. below rock benches marking the main floor on either side[3].

Along the sides of a recently cut valley mainly sound rock may be exposed or, if the rocks are less durable, they may be covered with a little soil and weathered material. Conversely, the depression of a region causes drowning of valleys near their mouths and, farther upstream, the slowing down or cessation of erosion and widespread deposition of alluvium. Again, a river for some reason or other may cut back along its headwaters and across the watershed separating its valley from that of a river flowing in another direction. In this way it will "capture" part of the channel and gathering ground of its neighbour, increasing thereby its volume of water and consequently power, for still more active erosion. Later in the river's history, the conditions for a dam-site may be most favourable where the former watershed existed.

The last-mentioned situation is illustrated by the investigation of Mr. Irving B. Crosby, consulting engineering geologist, of Boston, Mass., along the St. Maurice river, Quebec[4]. The region of hard granitic gneiss traversed by the river is largely covered with glacial deposits, with exposed rock at various places. The direction of flow of both the pre-glacial and post-glacial rivers is, or was, mainly eastwards. The upper part of the post-glacial St. Maurice river, however, followed a new course, cutting across drift-filled pre-

glacial valleys (Fig. 1). In fact, it was found that no less than four, and possibly five, pre-glacial river courses are crossed by the present river, which in at least three places also traverses old rock divides between pre-glacial valleys. These places, situated along a 35-mile stretch of the river, form ideal sites for dams, deep cuts in the rock having been excavated by the rapidly flowing water. Also the rock divides, extending on either side of the river, form, as it were, continuous dams beneath the drift deposits, except for the breaches along the present river-course itself.

Two examples concerned with metamorphic rocks will illustrate the part which time plays in the extent to which a rock is decomposed. One is in a ravine in the Southern Highlands of Scotland cut by a post-glacial river, the Finglas Water, where it is proposed to build a dam for augmenting the water-supply for Glasgow. The valley lies to the east of Loch Katrine, in the Trossachs, to which a tunnel will be driven from the Finglas beneath the dividing hills. Schistose rock is well exposed along the river bed at the proposed site and is either outcropping or covered with thin soil on the steep sides of the glen (Fig. 2). There is little superficial weathering of the schists although these rocks are in general among the less durable of metamorphic varieties. Had they been exposed for a prolonged time, there is no doubt that they would now be deeply weathered. This, at any rate, is the state of somewhat similar metamorphic rocks along the Little Tenessee River, North Carolina, lying just south of the farthest extension of the Glacial Period ice sheets of North America. Weathered rock and residual soil covered the sides of the river-valley to depths of 80 to 100 ft. at the excavation for the Fontana Dam (Fig. 3). Recent erosion along and adjacent to the river itself had fortunately left little bad rock there. At another site in North Carolina, the Hiwassee Dam, even worse conditions were present[5]. In addition to general decomposition of the relatively durable quartzite and mica-schist at this site and the formation of thick residual soils, the rocks were traversed by sheet-joints parallel to the surface, like the sheet joints prevalent in granite, both along the river-bed and at the abutments. Water had been passing along these fractures for a long time and had caused the complete decay of the adjoining rock. Also, still deeper decomposition had taken place along faults. The total volume of earth overburden amounted to nearly 65,000 cu. yards, and in addition a total of 294,500 cu. yards of rock had to be excavated.

Effects of Climate

Climatic conditions have their effect upon the weathering of rocks and transport of the weathered materials. For example, it is well known that rainfall in tropical regions and wind in arid climates are potent agents in removing soil and baring rock. Although soil erosion means disaster to the farmer it may be helpful to the engineer. In the case of recently glaciated country we are apt to forget what might have been and accept the bare rocks and scanty soils as a normal expression of Nature. Glacial erosion has indeed played a notable part in furthering hydro-electric developments. Through its action ideal topographical situations for both dams and reservoirs have been provided, with rock-ridges bounding rock-basins. An example from a granitic terrain in the Scottish Highlands, the site of the Gaur dam and rock-basin west of Loch Rannoch in Perthshire, illustrates the action of a recent arctic climate (Fig. 4).

The modes of formation and means of transport of weathered and eroded rock-materials vary with the climatic conditions. When deposited, the materials constitute transported "soils." These unconsolidated "drift" deposits of recent times were laid down for the most part on the land after transport by water, ice or wind. To water transport is due the formation of alluvial silts, sands and gravels along valley floors, with occasionally the accumulation of clays, sometimes of the calcareous variety, marl, on the floors of the lakes. To ice, we can assign that impervious, often tough deposit, boulder clay, and looser-textured moraine, as well as fluvio-glacial deposits of sand and gravel formed near the margins of melting ice-sheets and glaciers. To wind we can attribute deposits of fine sand and the impalpable hardened dust called loess. In recently uplifted regions marine deposits are found, forming raised beaches bordering the coasts. All these deposits, with the exception of those due to wind, are very liable to vary both in thickness and composition from place to place, and the geologist can seldom supply an assesment of them at a dam-site sufficiently accurately for the final requirements of the engineer. Still, from a study of the vicinity and the piecing together of evidence obtained at various exposures of the materials, he can usually obtain information upon which a general idea of the situation can be based.

Durability of Rocks

There is no greater variable than the degree of durability of rocks when subjected to the chemical action

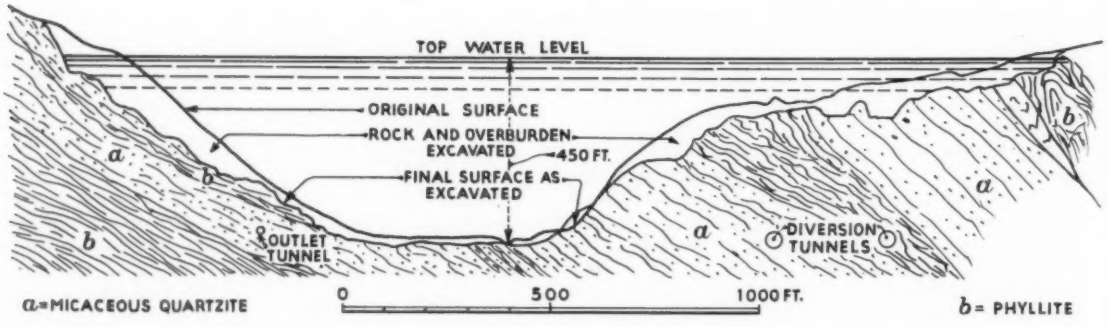

Fig. 3. Section across site of Fontana dam, North Carolina, U.S.A., showing thick "soils" covering sides of ancient valley. (After Plate 7, Burwell and Moneymaker, Berkley Volume. Geol. Soc. America (1950))

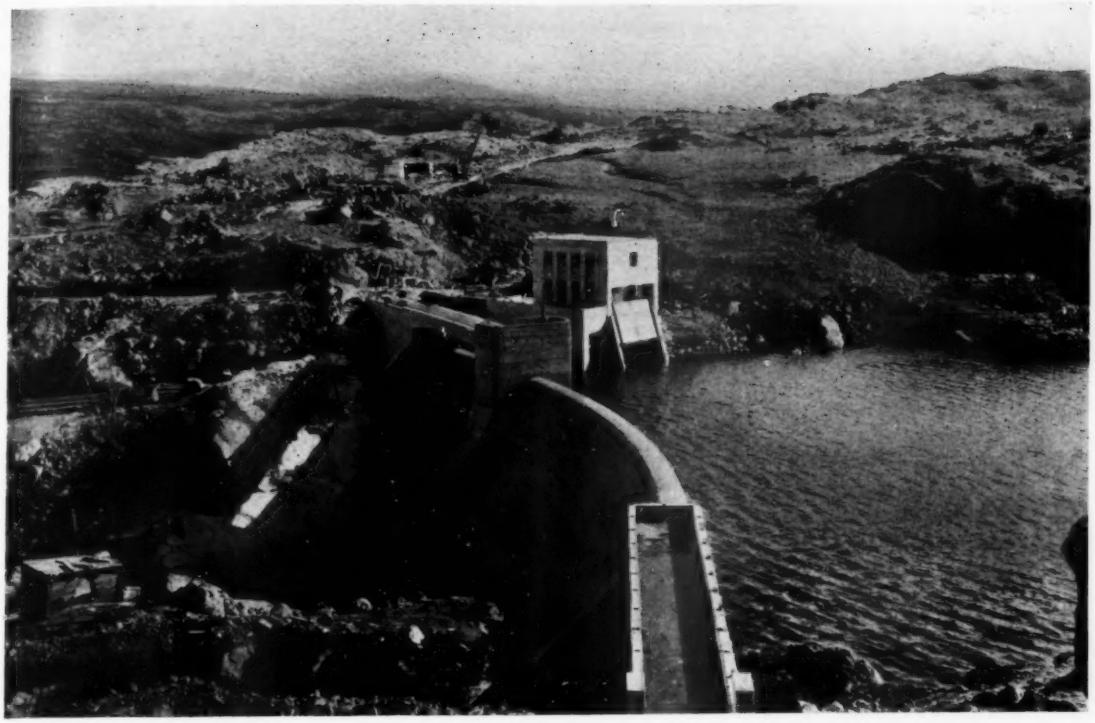

Fig. 4. Gaur dam, west of Loch Rannock,Perthshire, Scotland; this dam spans a river channel cut in post-glacial times across a broad granite rock bar (grandiorite) upstream of which the ice eroded a pronounced rock basin. (By courtesy of the Secretary, North of Scotland Hydro-Electric Board)

of percolating rain-water. Not only the rock itself but also the local situation of a rock have to be considered. On general grounds the presence of certain minerals in a rock may suggest caution. A mineral such as potash felspar and also the soda-lime variety are liable to be altered to clay, and calcite (calcium carbonate) is apt to be dissolved. Iron sulphide occurring in rocks as that brassy-coloured mineral, iron pyrites, may be decomposed and weak sulphuric acid formed. This acid then acts upon other minerals and brings about their dissolution, and the smaller and less noticeable little grains of pyrites are, the more readily are they affected. As regards the local situation, deeper decomposition is likely where water is constantly available, as along some valley-floors as compared with the valley-sides, or where rock is overlain by permeable "soil" as opposed to an impermeable clay like boulder clay. Where, however, boulder clay is thin, it may itself have been weathered and become permeable down to rock-head and the rock itself altered. A not infrequent situation is to find that this clay has been pressed into cracks in the rocks by the weight of the superincumbent ice, forming the clay-backs in an excavation.

On account of the many processes and situations bearing upon weathering, the direct method of assessing it will best take into account the engineer's point of view. Examination of the rocks at the actual site and in the vicinity, before boring, is most likely to provide the information required. Still, a particular example comes to the writer's mind, though only one of many, of excessively deep rock-decomposition which could not have been anticipated from any surface examination, although the main difficulty of the site was not concerned directly with rock-weathering. The construction of the Silent Valley dam in the granitic Mourne Mountains for impounding a reservoir for the water-supply of Belfast in Northern Ireland is well known to engineers on account of the great difficulties that were encountered and which arose from the occurrence of thick water-laden morainic sands and silts. At the excavation stage, deep bores were put down, into the granite, to depths far below the base of these deposits. During a visit to the site the writer was shown the bore-cores, and in these the granite was soft and friable throughout. It had evidently been decomposed to a great depth along the floor of the valley, although it was little affected above the moraine line on the valley sides. The reason for the deep decomposition may have been the hold-up of water by an impermeable barrier of the country rocks (Silurian shales) which, by inference, bounds the granite mass a short distance downstream from the dam. Even on hillsides in the Mourne Mountains, decomposition of fine-grained granite next to bounding "walls" of hardened shales is a recurring feature.

A final point concerned with durability of rocks is the question of scour at and below the spillway and toe of a dam. Much research is carried out by engineers working with large-scale models on the extent of the water jump, turbulent flow and other hydraulic problems at this vital place. All that need be said here is that geological conditions, especially

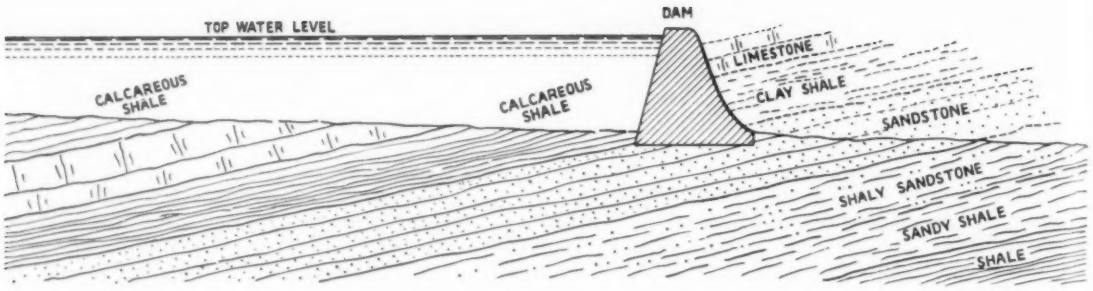

Fig. 5. Diagrammatic section of site with alternating groups of shale, sandstone and limestone rock, showing how an upstream dip of the strata often provides the best conditions for a dam site

the incidence of joints and other planes of weakness, have also to be investigated.

Strength of Foundations

In the case of a gravity dam, the strength of most unweathered rocks is usually adequate for meeting the compressive stresses they are required to bear. Exceptionally, alternations of stronger and weaker rocks, such as sandstone and soft, uncemented shale, or the latter rock where it occurs by itself, may entail much research and special measures, and it has often been the practice to excavate soft shale down, or near to, a sandstone, where this is practicable. With an arch type of dam the need becomes specially necessary for ascertaining the strength of the abutment rocks. As is well known, their modulus of elasticity should exceed that of the concrete. It is noteworthy that no instance of structural failure of an arch dam is on record[6].

In high dams, deformation of the foundations may result from the pressure exerted on the floor of the reservoir by the impounded water. The width of the reservoir will be increased very slightly, the amount of the increase depending upon the modulus of elasticity of the rocks lining the basin and the pressure exerted by the impounded water. In arch dams the effect of the deformation will be to cause a pull at the springing of the extrados of the arch and at the crown of the intrados. So in designing the dam this factor may have to be taken into consideration. In a gravity dam, the effect will be to widen the contraction joints. Recent investigations in Switzerland show that, for a dam 280 m. high and with a modulus of elasticity of the rock of 100 metric tons per sq. cm., the amount of the deformation to be allowed for is 20 to 25 mm.[7]

In the case of bedded rocks, upstream dip of the bedding planes is often desirable as, apart from cross-joints, strata are strongest in the direction across the bedding (Fig. 5). With horizontal beds, or those with a low dip downstream, slipping may take place along these planes of weakness unless the foundations of a dam are adequately inset in the rocks. The failure of the Austin dam, Texas, in 1900, was in part at least the result of the foundation rocks sliding in this way[8]. The dam, 60 ft. high and 66 ft. wide at the base, was founded on level-bedded Cretaceous limestone interbedded with clays and shales, and had stood for seven years. The clays and shales were not only slippery but also broken by a fault zone, and the limestones were jointed, cavernous and water-bearing. The disaster was attributed to slipping along the shales combined with uplift pressure; it occurred during a flood, the water coming over the crest scouring out part of the downstream toe. The dam was sheared through and two pieces of it, each about 250 ft. long, were moved bodily downstream.

Danger from shear along foundations is obviously present where weak parting planes in rocks, whether due to bedding, schistosity or jointing, are inclined in a direction across a site, and so are directed downslope at one of the abutments. Inadequate insetting of foundations at an abutment in such a case may lead to disaster, as happened at the St. Francis gravity concrete dam in California (Fig. 6). This striking failure also illustrates another feature which rarely requires consideration, namely, the part played by the durability of a rock as affecting its strength. Most rocks of adequate strength when fresh, will not fail through decomposition and disintegration during the time they

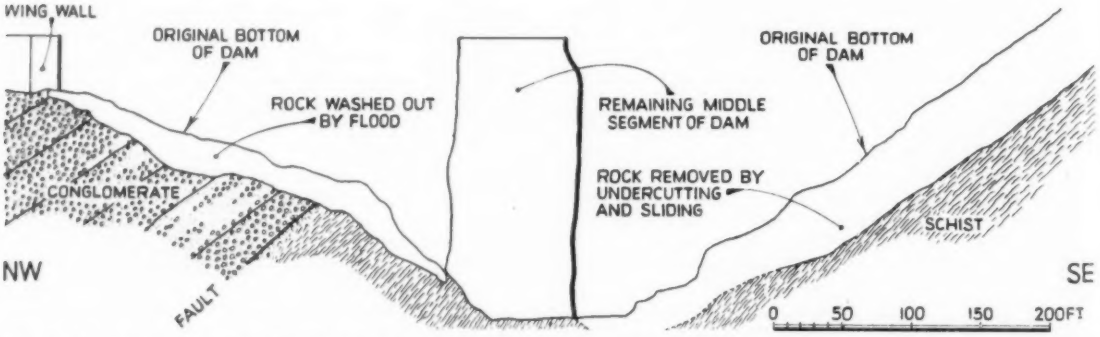

Fig. 6. Section across the site of St. Francis dam, California, after the failure of the structure in 1928. (From Fig. 3. F. L. Ransome, Econ. Geol. *23)*

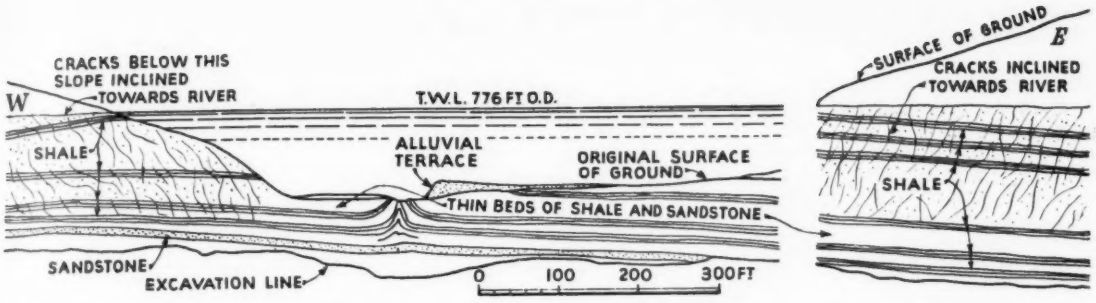

Fig. 7. Gravity creep of valley sides producing superficial making-up of shales at centre of valley and cracks inclined towards the valley floor on either side; Derwent dam, Derbyshire, England. (After E. Sandeman, Proc. Inst. C.E. 206 (1920))

will be required for engineering foundations. An exception was a conglomerate underlying one of the abutments of the St. Francis dam. Only a week after the reservoir had filled, in 1928, the dam gave way. Passage of water below the dam had been noted earlier, but it appears that the main cause of the failure lay with the conglomerate. This rock possessed a matrix of clay and was also finely veined with a soluble mineral, gypsum (hydrous calcium sulphate). After the disaster it was shown that though the rock was coherent when dry it rapidly disintegrated when placed in contact with water[9].

Comminuted and shattered rock along a fault is in general too weak to bear the weight of a gravity concrete dam, or the individual buttresses of a buttress dam. Part of the dam may have to be supported on an arch founded upon sound rock on either side of a fault-break. At the Rodriguez dam on the Tijuana, Mexico, a broad fault-zone occurred along the river-bed, and the adjacent rock was also broken and unsuitable for foundations. To meet these difficulties a concrete arch, founded on sound rock on either bank and having a span varying from 76 ft. to 86 ft., was first constructed across the river (Fig. 9). The arch was extended for 225 ft. along the river-gorge and upon it a buttress-type dam was constructed, 240 ft. in height above the river bed[10].

In another case, that of the Mattupatti dam, Travancore, India, a fault-zone 70 ft. in width extended along the river-bed[11]. The rocks consisted of highly-weathered pegmatites, gneisses, etc., traversed by zones of clay-gouge 4 ft. thick in all. The excavation was taken throughout the full width of 70 ft. to a depth of 130 ft. below the river-bed in order to obtain a suitable foundation. The height of the dam above the river-bed was less than the depth excavated, being about 125 ft. It should be mentioned that at this site rock-weathering extended for 50 ft. into the valley sides. The procedure adopted for the Rodriguez dam was presumably not practicable.

Leakage

Water passing through the foundations of a dam is a serious matter and every possible precaution is generally taken for preventing it or, if this is not practicable, for limiting its harmful effects. These may be summarised as follows:

(1) Undue loss of water from the reservoir.

(2) Uplift pressure exerted on the dam, especially at its deepest part where the hydrostatic pressure is greatest, by water entering the foundation rocks from the reservoir. The dam's capacity for withstanding the water pressure of the reservoir is thereby reduced, and in gravity dams there arises a danger of the structure being overturned. Uplift pressures, too, may cause cracks to develop across the structure.

(3) Existing channels in rocks may be enlarged by the passage of water through them, and in the case of "soil" foundations and earth dams the enlargements known as "piping" are of special concern. There are also instances of progressive reduction in the amount of leakage through such a rock as sandstone, owing to the sealing up of the pores and fissures by silt or clay carried in from the reservoir.

(4) Passage of water through fissures in rocks at the backs of abutments may reduce unduly the capacity of the rocks to resist shearing stress. It may also subject the rocks downstream of the dam, where they are not contained by the dam, to a hydrostatic pressure sufficient to bring about their disruption or to cause landslips, particularly where the water passes along fissures within, and parallel to, the slope.

Geological reasons for leakage in rocks are mainly the following:—

(1) Joints and faults extending across a dam site.

(2) Permeable strata such as varieties of sandstone, including where these beds occur below the foundation rocks.

(3) Cavernous rocks, especially solution cavities in limestone (Fig. 8).

(4) Easily soluble bedded deposits, gypsum, for example, even if they occur at some depth beneath foundations.

(5) Disturbed, broken and contorted rocks resulting from lateral pressure exerted by valley-sides; this is especially prevalent in incompetent rocks such as shales along valley floors (Fig. 7).

(6) Cracks parallel to slopes at the back of abutments, as in landslipped masses, or as the less extreme effects of gravity, especially in the case of steep-sided valleys (Fig. 9). This may occur even when the slopes are by no means pronounced. Of rather special origin are cracks formed usually in projecting spurs along valley sides in glaciated country, where ice has penetrated into the rocks and forced them apart. This is a kind of incipient "plucking" action, plucking being the usual way in which ice under pressure removes rock from its floor. The cracks may occur many yards in from rock-head, and are commonly filled with "soil." Infillings found at sites in the Scottish High-

lands usually consist of clay, sand, and interlaminated silt, sand and clay.

Difficulties During Excavation Work

Both advantages and disadvantages of an engineering site as regards its excavation may be pointed out by a geologist after his preliminary examination, although they are often not apparent until the site has been fully explored by means of bores, trial pits or headings. They are of especial importance at a dam, where unforeseen disadvantages can be a source of much extra trouble. Special means of dealing with

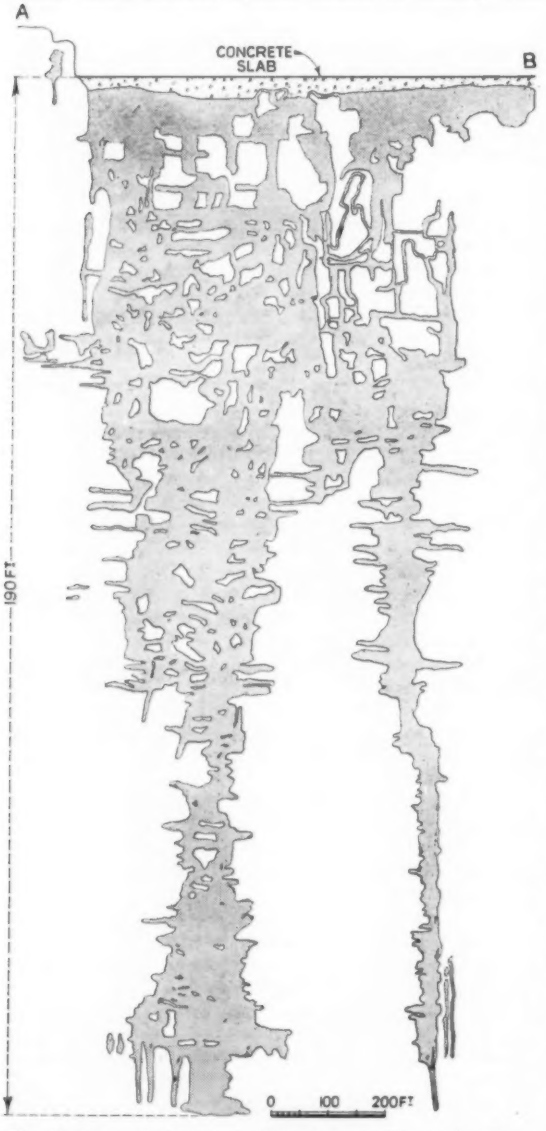

Fig. 8. Deep cavitation in limestone at Kentucky dam, Tennessee river, U.S.A. Solution of the limestone was mainly controlled by vertical joints but was also partly controlled by the bedding which is inclined at a slight angle to the right of the section. (Redrawn from Plate 4, Burwell & Moneymaker, Berkley Volume. Geol. Soc. Amer. (1950))

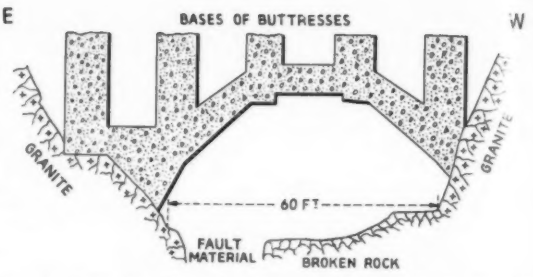

Fig. 9. Showing how a dam was founded to avoid weak fault rock by the construction of a supporting arch; Rodriguez buttress-type dam, Mexico. (Engineering News-Record, *105 (1930), 600)*

them may have to be devised, and the additional expenditure of time and money which some unpredicted geological feature can entail is not welcome in any engineering undertaking. Many difficulties are unpredictable. There are also many which detailed surface examination of a site and its environment, or a more thorough site-investigation, would certainly disclose. The conditions which might be encountered are too many to describe, and only somewhat general features can be referred to here.

Two main features to be considered are that which concerns the ease or difficulty with which the excavation work can be carried out, and the stability of the materials in the cut. Solid, massive rocks are in general reliable for excavating, examples being granite, many metamorphic rocks such as quartzose granulite, and hard sedimentary rocks like most sandstones and limestones. In bedded types it is well known that excavation is facilitated by the lie of the bedding being towards the working face. The disposition of joints is often of as much, or even more, importance. Shales and some phyllites (fine crystallised mica-schists) are often weak and for a variety of reasons may be troublesome to deal with. Spalling of uncemented, or poorly cemented, shales and mudstones has to be prevented by losing no time in covering them with concrete, or some other waterproof material, to prevent exposure to the weather. Bulging of these rocks where a superincumbent load of material has been removed is another recurring feature.

Excavation along the centre of a site or at the abutments may reduce the stability of the valley-sides, so that landslips may result. The possibility of land slipping has to be assessed with regard to the composition of the materials forming the valley-sides, both "soil" and rock, and their disposition. Planes of weakness inclined towards the river, whether bedding planes, joints, or contact-surfaces of "soil" with rock, have to be considered in relation to the materials concerned. A potential dangerous situation occurs where sandstone alternates with beds of shale and dips towards the river at a less angle than the slope of the valley-side. If the shales are of the uncemented types, they will become plastic when wetted, and overlying sandstone will slip downhill along the underlying shale. These conditions pertain, for example, to two dam sites in the Himalayas, on the Jhelum and Ramganga rivers, and for this reason the sites were condemned[11].

The ways in which underground water may occur

(Continued on page 389)

Effect of Gate-Valve Closure Characteristics on Water-Hammer Pressures

By R. SILVESTER, B.A., B.E., A.M.I.E.Aust.

In drawing the time-velocity closure curve of a gate valve for use in the graphical solution of water-hammer problems, an assumption is generally made that the coefficient of area of the valve remains constant (2/3) for all gate openings. Tests conducted at the University of Western Australia on a 4 in. gate valve have shown this to be incorrect. However, it is illustrated that the actual variation in this coefficient tends to make for the desirable linear closure.

WHEN a valve is closed on the end of a pipeline within a time $\frac{2L}{v}$ the maximum water-hammer pressure ensues. When the closure time is longer than this only some proportion of this maximum pressure is obtained. The pressure generally varies throughout the closing process. It can vary by oscillating about the value reached after time $\frac{2L}{v}$ or it can rise continuously until closure is complete.

SYMBOLS USED

A = area of pipe on which valve is installed (sq. ft.).
a = area of valve opening at any time (sq. ft.).
C = coefficient of area for valve.
d = diameter of line downstream of the valve (ft.).
f = Darcy's friction factor for line downstream of the valve (dimensionless).
h_v = friction head loss in valve (ft.).
L = length of line subject to water hammer (ft.).
l = length of line downstream of the valve (ft.).
V = velocity in pipe (ft./sec.).
v = velocity of pressure wave (ft./sec.).
Suffix $_o$ to any symbol indicates initial conditions, i.e., the valve fully open.

The type of pressure variation depends upon such factors as initial velocity and initial pressure in the line and also upon closure characteristics of the valve together with any impedances downstream of it. This latter is embodied in a time-velocity curve which gives the reduction in velocity with respect to time as measured from commencement of closure. If such a trace is a straight line one has *linear closure*, i.e., the velocity decreases directly with time. The antithesis is, of course, *non-linear closure*, and this can take either of two forms as illustrated in Fig. 1. All valves vary in some manner from the linear-closure curve. But since it has been shown[1] that linear closure tends to give the minimum water-hammer pressure, any phenomenon tending towards it is worthy of study.

The Time-Velocity Curve for a Valve

Two methods of obtaining such a curve have been outlined elsewhere[1], one of which involved the calculation of velocities from the area of opening of the valve, expressed as ratios in the form:

$$\frac{V}{V_o} = \frac{\left(\frac{A}{Ca_o} - 1\right)}{\left(\frac{A}{Ca} - 1\right)} \qquad (1)$$

In this equation it was assumed that C was constant at $\frac{2}{3}$.

Tests on 4 in. Valve

Since that time, tests have been conducted on a 4 in. John gate valve at the University of Western Australia to test the veracity of this assumption. The valve was installed on a 4 in. line with pressure gauges mounted to measure the head loss through the valve at various rates of flow for specific gate openings. By substituting in the expression

$$h_v = \frac{V^2}{2g}\left(\frac{A}{Ca} - 1\right)^2 \qquad (2)$$

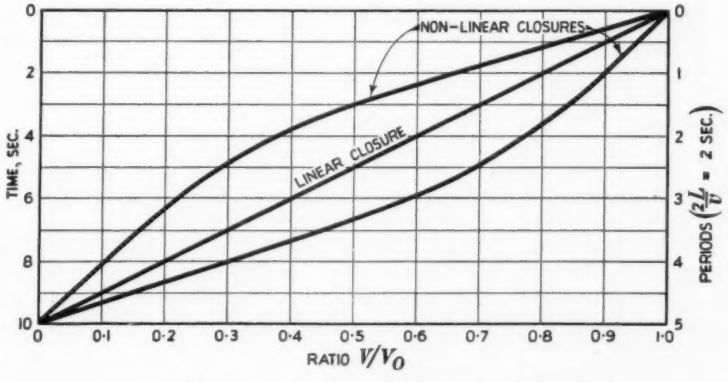

Fig. 1. Linear and non-linear closure characteristics

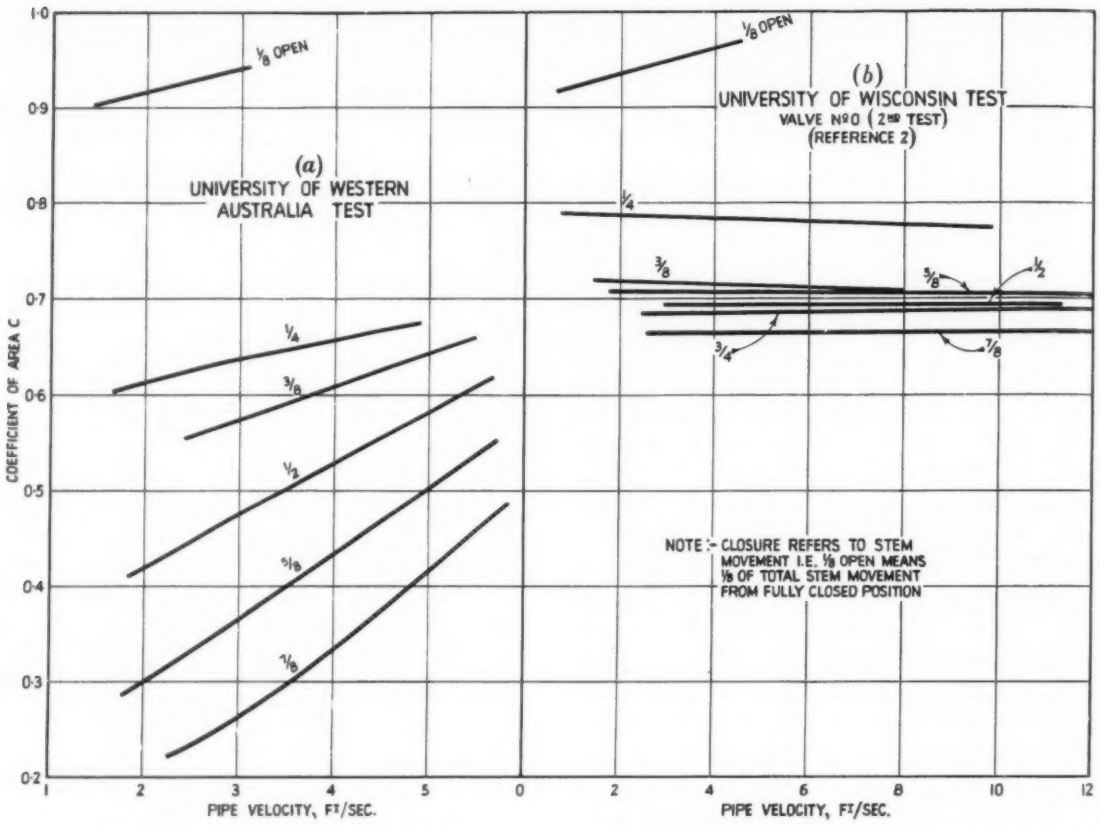

Fig. 2. Values of C obtained in tests at (a) *University of Western Australia, and* (b) *University of Wisconsin*

values of C were computed. A graph of the variables appears in Fig. 2a. From this it would appear that C varies with the velocity of the pipe.

At this time results of tests conducted on gate valves at the University of Wisconsin[2] came to hand.

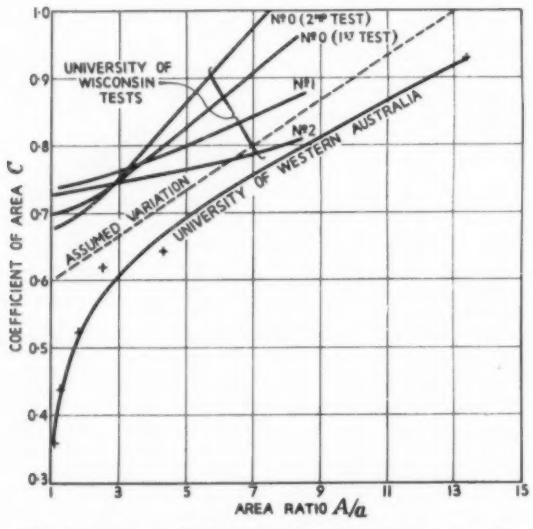

Fig. 3. Area coefficient C plotted against area ratio

These tests were carried out on several types and sizes of valve, but only the results on the three types of 4 in. gate valve were analysed to determine C. From this it was apparent that C remained almost constant over the range of velocities tested. Results for one of the valves only appears in Fig. 2b.

Now assuming C to remain constant for any gate opening at the mean for the velocity range used, it can be graphed against the area ratio A/a as in Fig. 3. From this figure it may be concluded that

(a) C varies with the area ratio A/a,
(b) the variation of C is not the same for different types of gate valve,
(c) for the gate valves in question, C varies from 0·35 to about 1·0, the main variation being 0·5 and 1·0.

Application to Closure Curve

If one assumes a range in C from 0·6 to 1·0 (as exampled in Fig. 3), the velocity-area curve can be drawn as in Fig. 4, by substitution in equation 2. Where a length of pipeline occurs downstream of the control valve the curve may be obtained by substituting the equation*

$$\frac{V}{V_o} = \sqrt{\frac{\left(\frac{A}{Ca_o} - 1\right)^2 + \frac{4fl}{d}}{\left(\frac{A}{Ca} - 1\right)^2 + \frac{4fl}{d}}} \qquad (3)$$

In Fig. 4 such a curve has been drawn for a line in which $\frac{4fl}{d} = 10$, for example $f = 0{\cdot}01$ and $\frac{l}{d} = 250$.

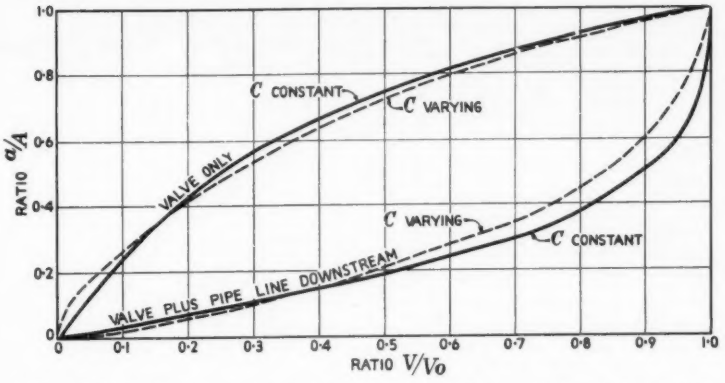

Fig. 4. Velocity-area curves for constant and for varying values of C

Conclusion

Since the $V/V_o - a/A$ curve in Fig. 4 is very similar to the time-velocity or closure curve for a gate valve[1] one can discuss its trends with this view in sight.

The important part of the closure curve for a gate valve, as far as water-hammer pressure is concerned, is at commencement of closure. It is readily seen that a variation in C (as assumed) tends to straighten this section of the closure curve computed by retaining C constant at $\frac{2}{3}$.

Thus the simplification of making C constant in the calculation of water-hammer pressures as outlined is a conservative procedure.

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* It is to be noted that this expression appears incorrectly as Equation 8 in reference No. 1.

From page 386

and may be tapped in an excavation are almost legion. A study of the local conditions, both in regard to soil and rock, is the only practicable way of assessing the possibilities within a limited area.

Finally, there is the matter of the reaction of rocks to drilling. Will the materials be very hard and require special drill bits? There are many cases where a careful local examination will be of help, not only in determining the types of rock which will be encountered, but also the details of their constitution. Relative hardness and toughness of rocks are qualities related to their reaction to the drill. For determining these two qualities, the British abrasion and impact tests, such as are made in laboratory investigations of roadstones, are therefore often of value to the driller. In regard to the special make-up of rocks, the sporadic occurrence of very resistant materials in otherwise easily drilled rocks is likely to introduce difficulty. There are many examples, such as hard pebbles in conglomerate, ironstone or other hard "balls" in shale, hard concretions (usually calcareous) in some sandstones, large portions of quartz in the coarsely crystallised granitic rock, pegmatite, and veins and blebs of quartz in schists.

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Los Peares Officially Opened

The Los Peares power station of the Fuerzas Electricas del Norveste S.A. (FENOSA) of Corunna was officially opened by General Franco on the 3rd of last month. This station is situated on the Mino, ten miles from Orense, in the north-west corner of Spain and is equipped with three generating sets each consisting of 62,400 kVA alternators vertically coupled to 72,800 h.p. Francis-type turbines manufactured by Boving & Co. Ltd. The generators, together with the other electrical equipment, were supplied by The British Thomson-Houston Co. Ltd., who have now issued a full technical description of the station and the associated works. This description not only deals with the Los Peares station as such but touches on the general power situation in Galicia, the factors which led to the choice of site, the design of the dam and the layout of the plant. It will be recalled that this station was the subject of an article published in the June issue of WATER POWER, 1952, at which time it was in course of construction.

The Shire Valley Project. It has been pointed out to us that the caption for Fig. 9, page 330, of the September issue should have read: Aerial view of Shire river at Chiromo in time of flood and showing the Ruo flowing in from the east.

Wheels that Turn No More

After more than one hundred years of continuous service, two famous water wheels have turned their last and will no longer provide power for The D.P. Battery Co. Ltd.'s factory at Bakewell in Derbyshire. Earlier this year a breakdown caused extensive damage and the firm has now reluctantly decided that it would be impracticable to consider trying to effect repairs. Such reluctance is understandable, for the wheels had come to be looked upon as one of the most interesting sights in the district, providing as they did a link with the earliest days of the industrial revolution.

The larger wheel, shown in the background of our illustration, dates from the year 1827 when it was installed to provide power for a cotton mill founded by the famous inventor, Sir Richard Arkwright, in 1777.

Built by the Manchester firm, Hewes and Wren, the wheel measures 25 ft. in diameter by 18 ft. wide, and has 70 buckets around its circumference, each capable of holding half a ton of water. Under normal working conditions the total weight of water in the buckets was about ten and three-quarter tons.

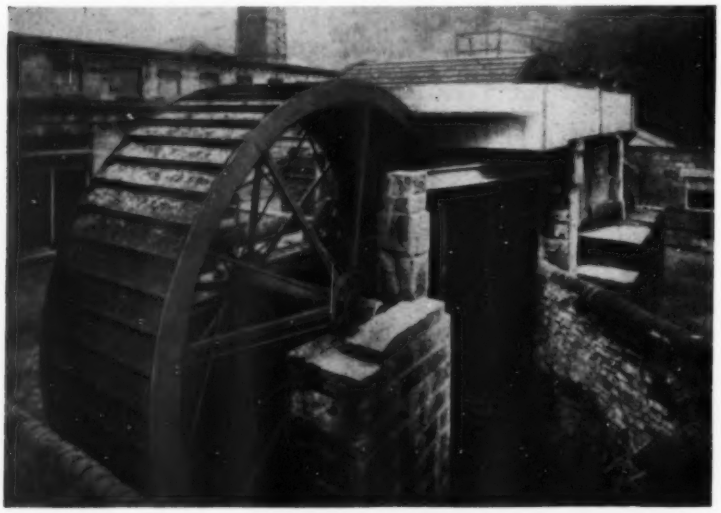
To carry such a weight, the wheel would have had to be of massive structure if the power were to be transmitted through the spokes and axle. A series of teeth were therefore arranged around the inner circumference of the wheel so that they engaged with a pinion situated just below the point at which the water entered the buckets. The weight was thus applied almost directly to the strong pinion wheel nearly 5 ft. in diameter and 1 ft. 2½ in. wide, which operated the shafting in the mill.

The smaller wheel dates from 1852. It was built by Kirkland and Son of Mansfield and measures 21 ft. in diameter by 7 ft. wide. The output of the two wheels together was between 60 and 80 kW.

Since the D.P. Battery Co. Ltd. took over the then disused cotton mill in 1898, the water wheels have been producing electricity used in the manufacture of storage batteries. Apart from their obvious usefulness for producing power, they have been or great sentimental value and during the last nine years £8,000 has been spent on repairs and replacements in the hope that they would continue to give good service for many years to come.

But this hope was shattered when one of the teeth on the large wheel fractured and jammed against the pinion. This resulted in many more teeth being stripped from the wheel and many of the cast-iron segments being broken. All the pinion wheel teeth were torn away and finally, the frame of the water wheel itself and of the driving wheel in the works cracked under the strain.

The wheels are now to be replaced by a modern water turbine.



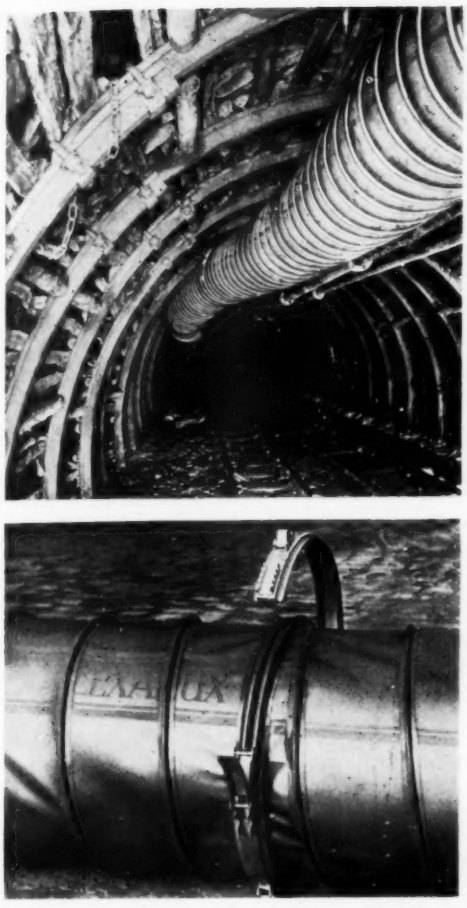
The waterwheels at the works of the D.P. Battery Co. Ltd., Bakewell, Derbyshire

Plastic Ventilation Ducting

The two principal types of ducting now in use to ventilate underground workings are made of metal or fabric. With the latter both transportation and installation are easier, but when stiffening rings are placed in the bore, the efficiency of the tubing is substantially lowered because of the high frictional losses. It is usually made of canvas fabric or woven jute, both of which are inflammable and liable to rot, although partial protection can be afforded by proofing with natural or synthetic rubber.

In an endeavour to overcome these disadvantages, Wilkinson Rubber Linatex Limited, Camberley, England, are marketing a plastic ducting under the trade name Flexadux. This has recently been developed on the Continent, where it is now extensively used. It is flexible and in its lightest form can be collapsed into so small a space that one man can carry 100 ft. With the exception of the steel coupling and stiffener rings, it consists of polyvinyl chloride plasticised with tricresyl phosphate. This material, which is 0·025 in. thick, is claimed to be proof against rot and not to support combustion; it has a tensile strength of 3,200 lb. per sq. in. It is said to be resistant to acid waters and suitable for use in temperatures ranging from 25 deg. C. below to 60 deg. C. above zero.

Due to their flexibility these ducts are stated to be relatively immune to damage, but should damage be sustained, a repair can readily be made by applying a simplified version of the same welding process that is used in the original manufacture of the material. The steel coupling and stiffener rings are totally enclosed by a plastic covering, and the stiffening rings are arranged externally on the duct so as to permit a smooth flow through the bore.



Flexadux plastic ducting, upper illustration, type B; lower illustration, type S

A patented hinge clip is used to ensure an airtight connection between adjacent lengths of ducting, this being seen in the lower of the accompanying illustrations. The ends of the ducts, which are reinforced with steel coupling rings, are placed in the clip and pressure is applied by means of an adjustable lever integral with the clip.

Flexadux is best installed by suspending it from either one or two guide ropes, and hooks are provided at regular intervals for this purpose. The guide ropes should be fixed to the roof supports about 25-30 ft. apart. Small thumb screws are used to anchor the hooks to the guide ropes after the ducts are fully extended. Where it is necessary to connect the ducting to either an existing metal duct or a fan casing, a connecting sleeve (either tapered or parallel) can be supplied to suit practically any requirement.

Three types of Flexadux are manufactured, type *S*, type *B* and the flat type. The first, seen in the lower illustration, incorporates a steel stiffening wire in continuous helical form, and is suitable for both positive and negative pressures up to 20 in. water gauge. Standard diameters are 8 in., 12 in., 16 in., 20 in., 24 in., 28 in., and 32 in. The standard length is 13 ft., although other lengths can be supplied.

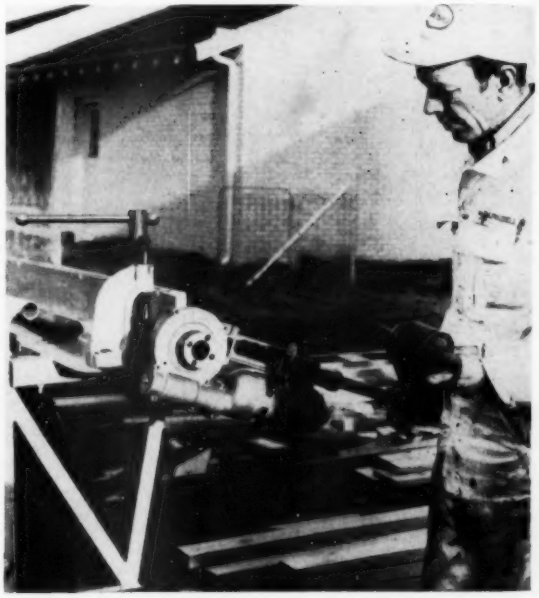
With type *B* (upper illustration) the steel stiffening rings are circular, this type being suitable for a positive pressure only up to 20 in. water gauge. Standard diameters are 8 in., 12 in., 16 in., 20 in. and 24 in., and the standard length is 13 ft., but, again, alternative lengths can be supplied.

The flat type is intended only for emergency use. No steel supporting rings are incorporated and the ducts are coupled together by interlocking the end coupling rings. Standard diameters are 8 in. and 12 in. and lengths can be supplied up to 150 ft.

Introduced four years ago, the installation of Flexadux in German and French mines is stated to be proceeding now at the rate of approximately a quarter of a million feet per annum. The Heinrich and Theodore Mine, at Essen, Western Germany, have prepared a report on its behaviour and in this it is stated that pressure loss per 100 m. length on metal ducting of 500 mm. diameter exceeded by 13 per cent. the loss from Flexadux of the same diameter when installed in the same ventilation line. Also the report claims that the quantity losses for the metal ducts varied from 5 per cent. to 27 per cent., depending on their condition, whereas the Flexadux showed a loss of only 0·5 per cent.

Portable Pipe Threader

The Velocity Power Tool Company, of Pittsburgh, U.S.A., are now manufacturing a portable power-driven pipe threader which weighs only 30 lb. and has an overall length of 27 in. This machine is marketed under the name Lawco Junior and is illustrated in the accompanying photograph. Single adapters and a speed reducer make it possible to handle pipe from $\frac{1}{4}$ in. up to 10 in. dia. The low weight and comfortable dimensions lend themselves to easy transport and it can readily be carried to where the threading is to be done. Also, it is so designed that it can be operated



Lawco Junior pipe threader

by one hand; all that is required is that the appropriate threading die should be fitted, the tool positioned and a button pressed to start the power unit This consists of a ¾ h.p. electric motor which is fitted with a toggle switch for reversing purposes. The tool may also be used as an auger, for boring post holes, for driving bolts and nuts, for opening valves, or even for hoisting weights up to 500 lb.

Rear Dump Wagon

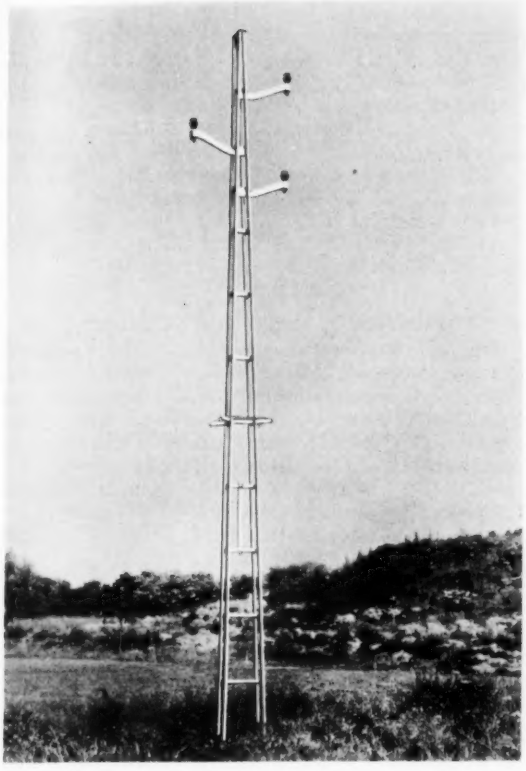
As was announced recently, The Birtley Co. Ltd. have entered into a manufacturing agreement with Athey Products Corporation of Chicago, U.S.A., and we are now informed that the first equipment of the Athey range to be brought into production at the Birtley works will be the PR. 21 rear dump wagon, which is designed to operate in association with the Caterpillar Tractor Company's DW. 21 wheeled tractor. This combination can haul loads of 28 tons at 20 m.p.h. The wide, open-topped hopper has a capacity of 22·5 cu. yards heaped on a 1 : 2 slope. The hopper provides an easy target for shovels, draglines or any type of loading equipment, and valuable time need not be lost in positioning it under the shovel. The hopper opening is 14 ft. × 9 ft. and a sturdy box-beam section around the top, sides and rear of the body allows lip loading without damage. The operator is completely protected by a wide shield running along the front of the body. This form of construction enables the PR. 21 to withstand the impact of big lumps of hard rock. The all-welded body has a three-layer bottom built up of ⅛ in. steel lower plate, an oak shock-absorbing layer and a ¾ in. steel top plate. Hydraulic hoists serve to tilt the trailer body to 60 deg. and provide fast, clean dumping well clear of the tyres. A heaped load can be ejected in 18 sec. Power for the hoists is provided by a pump situated on the rear of the tractor. The body pivots on the draftbeam and the tilting operation is effected by braking the rear wheels, when the tractor draws towards them; or, alternatively, by braking the front wheels the rear wheels are drawn towards the tractor as the load is dumped.

The wide track combined with a low centre of gravity provides exceptional stability. The tractor is powered by a six-cylinder CAT diesel engine of 275 h.p. and has five travelling speeds, ranging from 2·16 to 20 m.p.h. The hydraulic steering is positive at all speeds and 90 deg. turning gives non-stop turns in 34 ft. Timing gears assure a correct angle between the tractor and dump wagon. The weight of the tractor is 27,200 lb. and the dump wagon (unladen) 33,000 lb.

New Overhead Line Pole

A new overhead line pole for use on distribution and transmission lines where supports do not have to withstand heavy mechanical loading has been developed by Painter Brothers Ltd., of Hereford, in collaboration with British Insulated Callender's Construction Co. Ltd. It will be known as the P.N. pole. In its design full consideration has been given to the need for economy, lightness and simplicity of construction. These features have been achieved by fabricating the pole from standard commercial sizes of small-bore steel tubing and adopting a bracing system which permits a breakdown of the complete pole into component pieces. This, in turn, results in easy transportation and enables the assembly and erection of the

Birtley-Athey type PR. 21 rear dump wagon



Painter-BICC type PN overhead line pole

poles to be carried out with the minimum of skilled workmen and supervision.

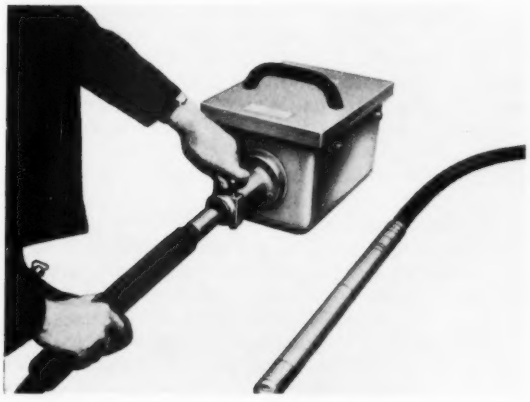
The pole is made in two sizes, 30 ft. and 35 ft. overall length, designed for a depth below ground level of 5 ft. and 5 ft. 6 in. respectively. The transverse face tapers from ground line to peak to form an "A" frame with a reverse taper below ground to provide an efficient foundation. Longitudinally the tubes are parallel.

The pole is made from four steel tubes of 1 in. nominal bore by 12G to Class A—B.S. 1387. The tubes forming each longitudinal face are connected at intervals by batten plates welded to the tubes and to these battens are bolted spacers, made from rolled-steel flats to form "I" sections, which connect the two longitudinal frames together to make the complete pole. The two longitudinal faces are fastened together at the top by a locking unit and a cap plate which are connected by a through bolt or insulator spindle. The cap plate also serves to seal the ends of the tubes from the atmosphere. All structural parts are hot-dip galvanised, bolts being hot-dip spun galvanised. The tubes are left open during the galvanising and are therefore protected both inside and outside. These poles are particularly suitable for tropical conditions. They have been designed for an ultimate head load not exceeding 1,250 lb. at the pole top, and have been subjected to full-scale comprehensive tests so that poles are of a guaranteed known strength. They will support a wide range of conductors on spans ranging between 250 ft. and 350 ft. depending on conductor size and material.

New Concrete Vibrator

A new concrete vibrator, known as the CCL-Tremix, has been placed on the market by Cable Covers Limited, St. Stephen's House, Westminster, S.W.1., the sponsors of the Gifford-Udall-CCL concrete prestressing system. It is of the electrically-driven immersion type giving 17,000 impulses per minute, obtained by centrifugal action from an impeller rotating at 3,000 r.p.m. The poker is only 1½ in. in diameter, so that it can be passed between reinforcement bars in circumstances where a large-diameter head would not be suitable.

The motor is mounted in a metal case complete with a pushbutton starter, an overload cut-out, and a protective device against wrong direction of rotation. A quick-action coupling enables the flexible drive to be attached or disconnected in a moment. The lubrication system is designed to give 12 months' service before re-oiling is necessary.



CCL-Tremix concrete vibrator

The total weight of the equipment is about 60 lb. A petrol-engine drive, running at 2,850 r.p.m., and a 2½ in. poker head are available as alternatives.

Electronic Air Cleaners. A series of illustrated leaflets issued by the Harris Engineering Co. Ltd., London, refers to the electronic cleaning units manufactured by this firm. Leaflet EP1 describes the "Trion" air cleaner for which a claim is made that it traps particles down to 100th of one micron in size (bacteria averaging about one micron); Leaflet EP2 refers to the Harris oil mist precipitator and Leaflet EP3 illustrates the various models of cleaning units now available.

Allweather Paints Limited. This firm has issued a new brochure designed to appeal to the technologist who is concerned with the protection of iron and steel, concrete, asbestos-cement, plaster, wood and other non-metallic surfaces. The effectiveness and durability of the firm's chlorinated-rubber paints are emphasised, and particulars are given of the Galvafroid/Pitan treatment of iron and steel to withstand severe corrosion conditions. An independent analyst's report is cited to substantiate the claims made for these protective coverings.

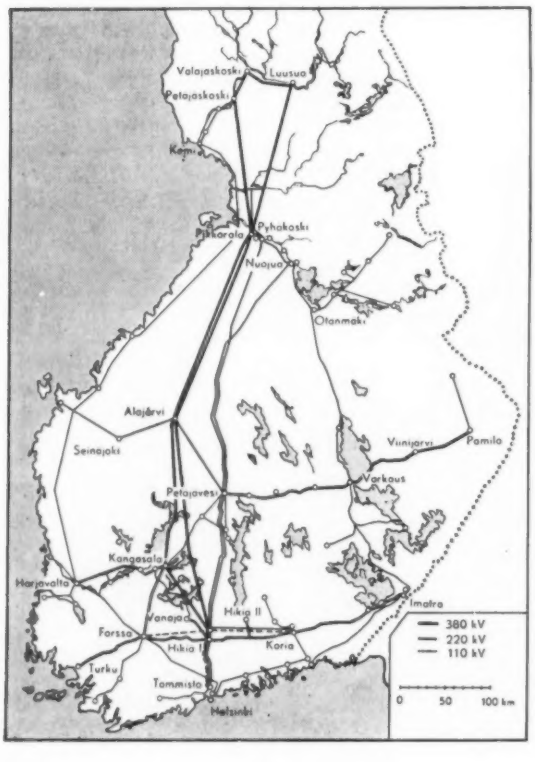
Abstracts from the World Technical Press

Electrical Testing of Weir Stability

This paper describes tests carried out by the electrical analogy method in an investigation into the foundation problems encountered in the construction of the Gandak Barrage in Bihar. Part I reviews the results obtained in investigating the influence of rocky strata on the distribution of pressure below a proposed barrage; part II deals with the stability tests applied to the Jobra Weir, across the Mahanadi, the crest of which it is proposed to raise from El. 67·55 to 69·5 in order to increase depth of the impounded water. This would naturally increase uplift and it was therefore important to test the stability of the structure under the new conditions proposed. (S. D. L. Luthra, *Irrigation and Power*, Vol. 12, No. 1, January, 1955, p. 102, 16 pp., 10 ff., 7 tables.)

Finland's New 380 kV System

Transmission problems are the same in Finland as in Sweden as in both countries there are vast power resources in the north and important consumption centres in the south. It is therefore quite natural that Finland should follow the example set by Sweden and add a 380 kV system to the present 110 and 220 kV lines. The accompanying map shows the three networks upon completion. Part of the 380 kV equipment is already on order; the ASEA concern will supply the Petäsjäskoski station with four auto-connected, single-phase, three-winding transformers of 40/85/125 MVA, as well as with a 70 MVA shunt reactor, 12·5 kV, with a STAL cooler. The 405 kV winding of the transformers is designed according to insulation class 1650 of the Swedish Standards; the system has a directly earthed neutral. The transformer oil is water and air cooled on the STAL system. Other orders include the switchgear, two airblast circuit breakers of the new V-Type and rated at 1,250 A and a breaking capacity of 12,000 MVA at 380-420 kV. (*ASEA Journal*, Vol. 28, No. 1/2, January-February, 1955, p. 38, 1 f.)



Pumped Storage in Macedonia

This article describes the Galicica hydro-electric pumping project which has been designed for lifting water 518 ft. from Ochrida lake, which straddles the Yugoslav-Albanian border, into Prespa lake, whose southern part lies in Greek territory. The hydrologic characteristics of those two lakes are as follows:—

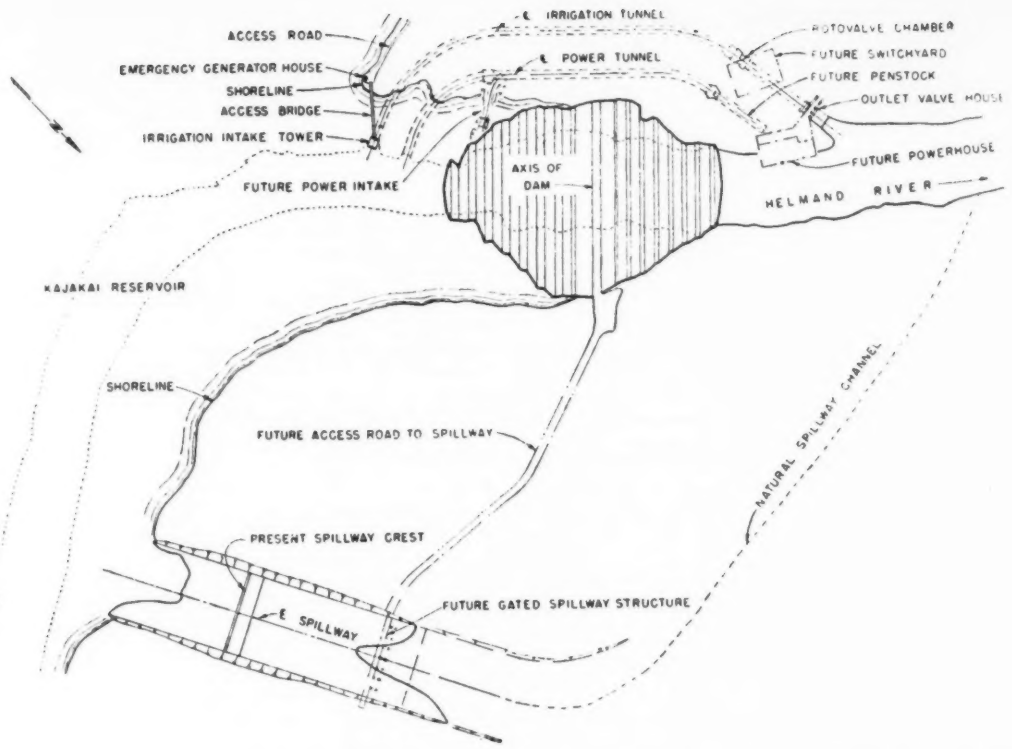
		Ochrida	Prespa
Catchment	(sq. km.)	977	1,334
Area	(sq. km.)	351	324
Yearly discharge (million cu. m.)		253	327

By diverting the Sateska river, a major tributary of the Black Drin (400 sq. km. catchment), into the Ochrida lake, the yearly discharge of the lake would be increased to 443 million cu. m. This additional storage capacity would raise or lower the water level of the two lakes by no more than 1·35 m. so that in practice they could be used as perennial reservoirs for power generation. For this reason, the Galicica plant, to be set up on the eastern shore of Ochrida lake, would be designed as a peak-period plant with a capacity of 50 to 60 MW corresponding to a discharge of 40 to 50 cu. m. per second, and operate but 2½ to 3 months a year at the time of maximum demand. Besides generation proper, the great volume of water available in the two lakes would give ample scope for a pumped storage plant, which would have similar hydraulic and electric characteristics to the upper stage of the Glockner-Kaprun system in Austria. Once the entire development of Macedonia was completed, there would be available, at week-ends, a surplus of energy which could be diverted to this plant.

The Galicica project is most certainly worth considering, as it would not only enable peak requirements to be met in the day time but also enable the normally wasted night and week-end energy to be conserved. A table appended to the article shows the aggregate river discharge and power production available in the whole of Macedonia. To mention production only, out of a possible total of 4,120 million kWh, the present figure from run-of-river plants at present in operation is no more than 80 million kWh, with the prospect of a further 400 million from storage plants now in course of construction. (Toma Marinov, *Elektrotehniški Vestnik*, Vol. XXIII, No. 1-2, 1955, p. 1, 4 pp., 5 ff.)

Helmand River Project

This multi-purpose project, the first of its kind in Afghanistan, is described at length. The earth and rockfill Kajakai dam, 320 ft. high by 900 ft. long between abutments, is sited in a short steep-walled gorge of the Helmand river. As the flanking hills are of adequate height, no auxiliary or saddle dams were



required. A natural dip in the ridge, about 2,300 ft. to the right of the dam, provided a suitable location for the spillway as well as for a natural escape channel which will carry the discharge well beyond the toe of the dam. Two diversion tunnels pierce the left abutment, one, at present in use, for irrigation releases, and the other to act as a supply tunnel for the future power house to be erected at its outlet. Present assessments contemplate a four-unit 120 MW plant and a minimum annual production of 250 million kWh. These figures are to be reviewed when more data on stream flow are available and demand for power justifies the construction of the plant. (H. W. Birkeland, *Indian Journal of Power & River Valley Development*, Vol. V, No. 2, February, 1955, p. 3, 100 pp., 12 ff.)

Bulkheads Solve Penstock Repair Problem

A brief account is given of the procedure applied to repairing the leaky penstocks at Horse Mesa dam, 65 miles north-east of Phoenix, Arizona, without losing water or interrupting generation. Draining the reservoir being ruled out, the engineering staff of Salt River Power District designed two bulkheads that could be slipped down the upstream face of the dam to seal off two of the penstock intakes, leaving the third in operation. The 8 ton bulkheads were fabricated of 12 in. wide-flange beams welded together, with the ends sealed off with steel plates and the face fitted with rubber gaskets to provide an effective water seal. Removing the badly-corroded 18 × 24 ft. trashrack, and subsequently lowering the bulkheads between the trashrack guides to the penstock openings near the bottom of the dam, was performed from a floating base which consisted of a sectionalised steel barge, whose 5 × 5 × 7 ft. sections were moved by land from Colorado to Horse Mesa reservoir, and reassembled on site. The third penstock was similarly repaired in turn .(*Engineering News-Record*, April 28, 1955, p. 43, 3 ff.)

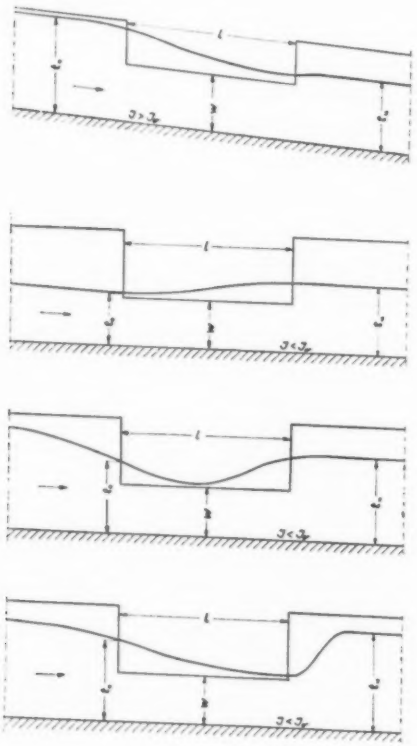
Prospects of Future Swiss Developments

A summary is given of two papers read at the general meeting of the Swiss Union of Energy Consumers, held in Zurich on March 23, 1955. Mr. Osterhaus, Director of the Swiss Federal Water Board, considers the question from the aspect of the various problems which will be encountered in future developments. Since 1947, the aggregate production of Swiss hydro-electric has increased at the rate of 500 million kWh p.a., against 300 million kWh during, and 200 million before, World War II. During this time consumption was growing at so great a rate that only water flows above the average enabled requirements to be met. If, however, future demand is to be covered, all technically utilisable water resources must be harnessed without priority being given to the apparently more economical projects. The author is of opinion that within the six coming years a stage can be reached when a shortage of electrical energy will become impossible. On the basis of present conditions, it is expected that all technically utilisable resources will have been fully developed by about 1980, and this would mean a yearly production of 30,000 million kWh and an increase in capital expenditure from 2,830 to 9,240 million Sw.Fr. Friends of nature need not be alarmed by this gigantic total development since it involves no more than a fifth or a quarter of all theoretically available resources. The second paper, read by M. Jaberg, is devoted to the financial aspects of the question. M.

Jaberg is against development at too fast a rate; it would not be wise, he considers, to force the present rate of initiating developments which could be advantageously carried into effect at a later date, and that it would be a much sounder policy, from the aspect of employment, to keep some developments in reserve for the future. (*Bulletin de l'Association Suisse des Electriciens*, Vol. 46, No. 8, 16.4.1955, p. 386, 2 pp.)

Calculating Side Weirs

When calculating the discharging capacity of side weirs, the problem generally presents itself as follows: In a channel carrying a variable flow, a determined water level is not to be exceeded downstream from a determined point. For given dimensions of a channel, it is easy to calculate the flow downstream of the weir, and if the flow approaching the weir is known, the length of the weir must be such as to enable it to discharge the difference between upstream and downstream flow. The author, reviewing the most important papers published on the subject, rejects Engel's widely accepted formula as proceeding from wrong premises and likely to give unreliable results; this formula varies according as the weir is set up in a section of the channel in which both banks are parallel, or at a point where either the weir or the corresponding part of the opposite bank is set at a slant, thus narrowing down the downstream reach of the channel. The method of calculation advocated by the author varies in accordance with the four patterns of flow shown in the appended diagrams, and, for the most frequent case occurring in practice of a channel with streamline outflow and purely streamline inflow towards the crest of the weir, a method applicable within fixed limits is suggested. Under other flow conditions, such as, for example, a partly turbulent flow towards the crest of the weir, model tests are indicated. The layout of a side weir at a slant or opposite a slanting section of the other bank is advisable only in cases where greater flow velocities may cause erosion in the channel bed. (Dr. Ing. Martin Schmidt, *Die Wasserwirtschaft*, Vol. 45, No. 4, January, 1955, p. 96, 5 pp., 7 ff.)



Lake Assal Hydro-Solar Scheme

Under reference to M. L. Vadot's article in *La Houille Blanche* (WATER POWER, April, 1955, p. 157/158), the possibility of harnessing Lake Assal to the generation of hydro-solar power, and other modes of utilising the lake, are discussed. After a critical review of all the information gathered on the useful evaporation area of the lake, the rate of evaporation, etc., the author arrives at the conclusion that to use the lake for the generation of hydro-solar energy as was suggested by M. Vadot, is unlikely to be practicable. He advocates a thorough reinvestigation of the now 20-year-old Gandillon project of a tidal development which would utilise the uncommonly strong currents (nine knots) of the Gulf of Tajura. The energy of the tidal plant would be absorbed by chemical works which could exploit the immense resources of magnesium salts, potash, bromine and rubidium which exist in the lake. In magnesium salts alone it is estimated that these resources represent the equivalent of 120 million tons of magnesium metal. (L. Legrain, Consulting Engineer, *La Houille Blanche*, Vol. 10, No. 2, March-April, 1955, p. 207, 4 pp., 1 table.)

CLASSIFIED ADVERTISEMENTS

Announcements for this column can be accepted up to the 8th of the month for the following month's issue. The charges are fourpence per word with a minimum of 10s. Box No. facilities 2s. 6d. extra. In order to avoid accountancy it would be appreciated if instructions to insert were accompanied with remittance.

Appointments Vacant

CIVIL ENGINEER required, aged 25-30, for structural and hydraulic design in London Office. Salary £1,000 p.a., good prospects; compulsory pension fund. Write stating qualifications and experience to Box W.P. 846 c/o 191, Gresham House, London, E.C.2.

CIVIL ENGINEER required, aged 25-30, for survey and construction work on hydro-electric development in KENYA and TANGANYIKA. Starting salary £1,200 p.a., good prospects. Length of Tour varies from 3 to 4 years according to altitude. One month's home leave on full pay for each completed year of Tour. Free furnished quarters when in construction camp or on Safari. Air passages free each way. Married quarters provided with wife's fare after satisfactory probationary period. Write stating qualifications and experience to Box W.P. 848 c/o 191, Gresham House, London, E.C.2.

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Applications are invited for a position of SENIOR LECTURER or LECTURER IN CIVIL ENGINEERING.

Applicants should preferably possess experience in the design and construction of hydraulic works and apparatus.

The salary ranges will be at least the following:—

Senior Lecturer ... £A1,750 to £A2,000 per annum
Lecturer ... ... £A1,300 to £A1,650 per annum

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Further particulars and information as to the method of application may be obtained from the Secretary, Association of Universities of the British Commonwealth, 36, Gordon Square, London, W.C.1.

The closing date for the receipt of applications, in Australia and London, is October 31st, 1955.